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DEPARTMENT OF COMMERCE

TECHNOLOGIC PAPERS

OF THE

BUREAU OF STANDARDS

S. W. STRATTON, DIRECTOR

No. 173

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BY

W. A. SLATER, Engineer Physicist
Bureau of Standards

F. E. RICHART, Structural Engineer
G. G. SCOFIELD, Structural Engineer
Emergency Fleet Corporation

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I. INTRODUCTION

1. PURPOSE AND SCOPE

This paper embodies the results of three somewhat dissimilar investigations which were made by the concrete ship section of the Emergency Fleet Corporation on the bond resistance or adhesion between concrete and reinforcing steel. One investigation was made to study the effect on bond resistance of the applications of various anticorrosive coatings on reinforcement. The second investigation was made to study the length of lap required for effective splicing of reinforcing bars in regions of high tensile stress. The third investigation was made to study the relative merits of different methods of anchoring the ends of stirrups to meet certain conditions which arise in concrete ship construction.

One of the first problems which presented itself in the design of reinforced concrete ships was to determine whether the reinforcement in the shell of a ship would be subject to corrosion, and if so, to devise methods of preventing such action. Corrosion would be dangerous because of the bursting of the concrete due to the formation of a bulky iron compound on the surface of reinforcing

bars rather than because of injury to the reinforcement itself. Experience with existing reinforced concrete structures exposed to the action of sea water or sea air has shown that a thin covering of ordinary concrete can not be expected to protect steel from corrosion indefinitely.[1] It was anticipated that this action might be prevented in a number of ways, as, for instance, (1) by using a waterproofing paint on the surface of the concrete; (2) by increasing the waterproofing qualities of the concrete itself; or (3) by using some form of protective coating on the reinforcement.

With regard to the first investigation, it was considered probable that the use of a coating on reinforcing bars would affect the bond resistance between the concrete and the bars. Hence, when tests were made to determine the effectiveness of various coatings in preventing the corrosion of steel, tests were also made to determine the effect of the same coatings on the bond resistance which could be developed. These tests were made on pull-out specimens and will be described in Section II of this paper.

The second investigation described in this paper was made because the designer needs definite knowledge as to the effectiveness of lapped bars as a tension splice. In lapping bands of reinforcement, it is important to know how many bars of a band may be safely lapped at one section, and how the length of embedment should vary with variation in the ratio of the cross-sectional area of the lapped bars to the total cross-sectional area of the steel. This is essentially a question of bond resistance, since it involves intensity and distribution of bond stresses along the length of embedment of the lapped bar.

Most of the available data on bond resistance are based on pull-out tests similar to those reported in Section II, in which the concrete block is in compression. Such tests have been shown [2] to agree reasonably well at corresponding slips with bond tests of reinforced concrete beams in which the ends of the reinforcing bars were embedded in a region either of compressive stress or of low tensile stress.

On the other hand, pull-out specimens in which the concrete block was in tension have shown [2] lower bond stresses than those specimens in which the block was in compression, and it seemed probable that bars ending in a region of high tensile stress would develop comparatively low bond stresses. To obtain evidence on

[1] R. J. Wig and L. R. Ferguson, What is the Trouble with Concrete in Sea Water, Eng. News Record, Sept. 20 to Oct. 18, 1917.

[2] D. A. Abrams, Tests of Bond Between Concrete and Steel, Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

this point test beams were made in which the tension reinforcement was lapped within a region of constant bending moment. The results of these tests are presented in Section III.

In the case of the concrete tank ship, the horizontal pressure of the cargo tends to force the shell outward from the frame, thereby causing tension in the stirrups which extend from the frame into the shell. The proper anchorage of the stirrups in a thin shell is a difficult problem, and it was to furnish information on this subject that the third investigation, as described in Section IV, was carried out.

2. ACKNOWLEDGMENT

The pull-out tests discussed in Section II were made at the concrete laboratory of the Bureau of Public Roads, at Arlington, Va. The tests were made between April, 1918, and January, 1919. A. T. Goldbeck, engineer of tests of the Bureau of Public Roads, was in direct charge of the investigation. R. Harsh and W. E. Rosengarten made most of the tests. The cooperation of the Bureau of Public Roads and of these men personally was of great value in the carrying out of the research work of the concrete ship section.

The beam tests of Section III were made at the John Fritz Laboratory of the Civil Engineering Department, Lehigh University, in December, 1918. Acknowledgment is made to Prof. F. P. McKibben and other members of the faculty for the facilities provided and for their interest in the work.

The pull-out tests of stirrups discussed in Section IV were made in the testing laboratory at the University of Pennsylvania in October, 1919, under the direction of S. C. Hollister of the Emergency Fleet Corporation. The facilities of the laboratory were made available for this investigation by arrangement with Prof. H. C. Berry.

Acknowledgment is made to A. S. Merrill, consulting concrete engineer of the Bureau of Standards, who has reviewed this paper. As a result of Mr. Merrill's work many portions of the paper have been modified, and some portions have been entirely reconstructed.

II. PULL-OUT TESTS WITH BARS TREATED TO PREVENT CORROSION

1. PRELIMINARY

This group of tests was made primarily to study the effect of the presence of anticorrosive coatings upon the bond resistance of the bars. A very complete series of tests made by Abrams[3] on uncoated bars was of much value in determining the type of specimen to be used, the technique of performing the tests, and the elimination of many undesirable variables which might have obscured the information sought.

The pull-out type of specimen used here has many advantages when it is desired to study a large number of variables. The specimen is easily and accurately made to any desired dimensions, can be tested quickly in a small testing machine, is fairly cheap, and gives results in usual terms with little calculation. For a comparative study like that reported here, it is an excellent type of specimen.

2. DESCRIPTION OF SPECIMEN

The pull-out specimen adopted for this work consisted of a one-half-inch steel reinforcing bar, 24 inches long, embedded axially in a concrete cylinder 6 inches high and 6 inches in diameter. In the position of molding and again in testing, the bar projected about one-half inch above the top of the cylinder and 17½ inches below the bottom.

In a total number of about 350 specimens, 18 kinds of protective coating were investigated. With a few exceptions, a set of 16 specimens was made with each kind of coating, and also with uncoated bars. In each set of specimens, 8 were made with plain square bars and 8 with deformed bars. Each group of 8 specimens was in turn divided into 4 pairs, which were stored under as many different conditions. Plain bars, except in series involving the use of metallic coatings, were painted by hand with a brush, while deformed bars were coated usually by dipping them into the paint. To supplement information obtained from these specimens, additional specimens were made in order to study the effect of varying the manner of application of certain paints.

[3] Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

3. ANTICORROSIVE COATINGS

A list of the coatings used on the reinforcing bars in these tests is presented in Table 1. This table furnishes the key to the system of symbols used for distinguishing all of the test specimens. A complete specimen number indicates the series number of this investigation in the research program of the concrete ship section, the kind of coating, the kind of bar used and method of applying the coating, and the method of storage. For example, 8HA1 is the specimen number given to a pull-out specimen in series 8, in which an asphaltic coating, H, was applied to a plain square bar by means of a brush, after which the specimen 1 was stored in damp sand for 28 days before testing.

TABLE 1.—System of Reference Symbols Used to Indicate the Nature of the Coatings, Methods of Application, and Conditions of Storage

[Investigation series No. 8]

Kind of coating		Method of applying coating			Method of storage	
First reference letter	Composition	Second reference letter	Kind of bar	How applied	Final reference numeral	Conditions
A	No coating used.	A	Plain square	Painted with brush.	1 and 2	Damp sand, 28 days.
B	Coal-tar pitch, coal-tar naphtha.	B	Deformed square	Dipped.	3 and 4	Damp sand, 1 month, salt water, 5 months.
C	Asphalt paint.	C	do	Painted with brush.	5 and 6	Damp sand, 1 month; salt water and air, 5 months.
D	Do.	D	Plain square	Painted and sanded.	7 and 8	Damp sand, 1 month; air, 5 months.
E	Iron oxide, siliceous material, varnish, turpentine.	E	do	Dipped.		
F	Asphalt paint.					
G	Do.					
H	Do.					
I	Iron oxide, siliceous material, varnish, mineral spirits.					
J	Red lead, linseed oil dryer.					
K	Iron oxide, calcium sulphate, siliceous material, varnish, mineral spirits.					
L	Iron oxide, siliceous material, varnish, mineral spirits.					
M	Asphalt gilsonite paint.					
N	Coal-tar drying oil, petroleum distillate, turpentine.					
O	Zinc (galvanized).					
P	Zinc (sherardized).					
Q	Zinc (metal spray).					
V	Phosphate coating.					
X	Electrogalvanized.					

Table 2 indicates schematically the exact classification of all specimens tested.

TABLE 2.—Classification of Pull-Out Test Specimens

[The numerals indicate the number of specimens tested]

Reference letter	Coating: Kind	Storage: Damp sand, 28 days, Specimens 1 and 2					Damp sand, 1 month Salt water, 5 months, Specimens 3 and 4					Damp sand, 1 month Salt water and air, 5 months, Specimens 5 and 6					Damp sand, 1 month Air, 5 months, Specimens 7 and 8				
		Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped	Plain painted	Deformed dipped	Deformed painted	Plain painted and sanded	Plain dipped
		A	B	C	D	E	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
A	(No coating used)	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	..
B	Coal tar	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	..
C	Asphaltic	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...	2	2	2	2	..
D	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
E	Ferric oxide	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	..
F	Asphaltic	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
G	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
H	.....do	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...	2	2	2	2	...
I	Ferric oxide	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
J	Red lead	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
K	Ferric oxide	2	...	...	2	2	2	...	...	2	2	2	...	...	2	2	2	...	...	2	2
L	.....do	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
M	Asphaltic	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...	2	2	...	...	...
N	Coal tar	2	2	...	2	2	2	2	...	...	...	2	2	...	2	2	2	2	...	2	2
O	Galvanized	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	...
P	Sherardized	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	1	1	...	...	...
Q	Metal spray	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	..
V	Phosphate film	2	2	...	...	...	2	2	...	...	...	...	...	...	...	...	2	2	...	...	..
X	Electro galvanized	2	...	...	...	...	2	...	...	...	...	2	...	...	...	...	2	...	...	...	...

4. MATERIALS

The aggregate used in making the specimens was a Potomac River sand, consisting principally of angular quartz grains and containing a slight amount of mica and clay. It was obtained in Arlington, Va. All of this sand passed a one-fourth-inch screen. A mechanical analysis made with a set of Howard & Morse sieves is presented in Table 3. The average tensile strength of three 1:3 cement mortar briquets was 250 pounds per square inch at seven days, which was 3 per cent higher than the average strength of similar briquets made with standard Ottawa sand.

TABLE 3.—Mechanical Analysis of Sand

Screen number	Separation size	Per cent passing	Screen number	Separation size	Per cent passing
	Inch			Inch	
¼-inch	0.250	100.0	50-mesh	0.011	24.3
10-mesh	.075	92.3	80-mesh	.0068	11.6
20-mesh	.034	76.1	100-mesh	.0055	7.5
30-mesh	.022	52.0	200-mesh	.0029	2.8
40-mesh	.015	35.2			

Tidewater Portland cement, having a fineness such that 83.9 per cent passed a No. 200 sieve, was used in making the concrete. In the concrete the proportions by weight were 1 part cement, 2 parts sand, and 0.485 parts water. With these proportions the water was 16.2 per cent (by weight) of all dry materials and 75 per cent (by volume) of the cement. Defined by sizes of aggregate, this mix was a mortar instead of a concrete, but, following the usage developed in connection with early concrete ship work, it is here termed concrete.

The steel used consisted of one-half-inch plain square bars and deformed bars of the same nominal cross section and perimeter.[4] The deformed bar was of the form shown in Fig. 1. It had nine lugs per foot of length. The lugs were truncated wedges about 0.05 inch in height, and included an angle of about 40° between the beveled bearing faces. It is evident that this form of lug gave some wedging action in the pull-out test, as verified by the fact that all pull-out specimens made with deformed bars failed by bursting the concrete block. No lateral reinforcement was used to restrain the concrete.

Physical properties of the steel used are given in Table 4.

TABLE 4.—Physical Properties of Steel

Type of bar	Yield point	Ultimate tensile strength	Ultimate elongation in 8 inches
	Lbs./in.²	Lbs./in.²	Per cent
½ inch, plain square	33 150	46 200	14.8
½ inch, deformed	61 300	105 750	16.6

In a few specimens the yield point of the bar was exceeded before failure occurred.

[4] A measurement of one plain bar and one deformed bar showed the perimeter of the deformed bar to be about 4 per cent less than that of the plain bar.

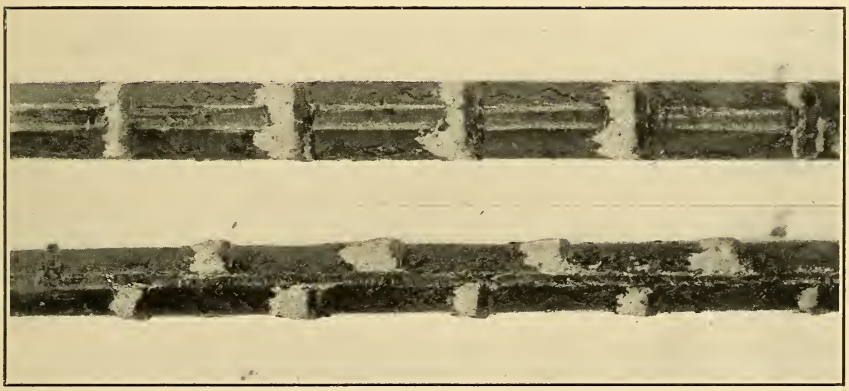


FIG. 1.—*Deformed bar used in pull-out tests*

5. MAKING OF SPECIMENS

The bars were prepared several days before the molding of the specimens. The plain bars were fairly smooth and had but little rust, and that was removed with emery cloth. One coat of paint was applied with a varnish brush over one-half of the length of each bar painted. The paint was allowed to dry for several days.

The deformed bars had a heavy coating of mill scale and rust. This coating was removed by scraping with the edge of a cold chisel and then brushing with a wire brush. Each bar was held vertically and the lower half of its length was dipped into the paint. The excess paint was allowed to drain off, and the bar was left to dry for several days. All paints were applied cold.

To compare the effect of different methods of applying the coating, a few specimens were made in which the plain bars were dipped and the deformed bars were painted with the brush. A few of the plain painted bars were also coated with a sharp fine sand immediately after painting, to study the effect of such a treatment.

Obviously, the above description does not refer to the application of metallic coatings, which were applied by commercial methods by the manufacturers to whom bars were furnished for such treatment.

In making the specimens, the cylinder molds were placed on a wooden bench. Holes (three-fourths inch in diameter) in the bench permitted the bars to be placed vertically in the axis of the cylinder, with the coated end projecting about one-half inch above the top of the mold. The bars were held in place in the bench by packing cotton waste tightly around them.

The concrete was tamped into place in the mold with a small wooden rod, and the top surface was smoothed off with a trowel.

6. AUXILIARY SPECIMENS

Three compression cylinders, 2 inches in diameter and 4 inches high, were made from the batches of concrete mixed for the pull-out test specimens. Average compressive strengths as determined for the different conditions of age and storage are as follows:

	Lbs./in.^{2}
Damp sand, 28 days	5090
Damp sand, 6 months	6195
Damp sand, 28 days, and sea water, 5 months	7010

7. STORAGE OF SPECIMENS

The specimens were removed from the molds when 24 hours old and were stored in damp sand in the laboratory for approximately one month. After this preliminary curing, different groups of specimens were accorded different treatment. In all cases specimens were allowed to stand in air 24 hours previous to testing.

Specimens 1 and 2 of each group were removed from the damp sand after 27 days and were tested on the following day. Specimens 3 and 4 were removed from the sand after 1 month and stored in artificial sea water for 5 months, then allowed to dry for 24 hours before testing.

Specimens 5 and 6 were removed from the sand when 1 month old and were stored in artificial sea water and in air alternately 12 hours each day for 5 months and then tested. These specimens were stored on a rack arranged to be readily lowered into and raised from the tank containing the sea water.

Specimens 7 and 8 were removed from the damp sand when 1 month old, stored in the laboratory uncovered for 5 months, and then tested.

All specimens were stored inside the laboratory where the range of temperatures was between 60 and 75° F. There is no indication that these temperatures produced harmful effects on any of the paint coatings.

The storage in sea water was to determine how the bond strength might be affected by the deterioration of paint and metallic coatings and by corrosion of the bars.

The artificial sea water referred to was prepared from the formula given in Table 5, based upon a typical chemical analysis of ocean water.

TABLE 5.—Composition of Artificial Sea Water

Chemical name	Symbol	Percentage composition of solids	Parts by weight
Potassium bromid	KBr	0.29	1
Potassium sulphate	K_2SO_4	2.55	9
Calcium sulphate	$CaSO_4$	3.97	14
Magnesium sulphate	$MgSO_4$	4.12	[a] 14.5
Magnesium chlorid	$MgCl_2$	10.78	38
Magnesium carbonate	$MgCO_3$	.29	1
Sodium chlorid	NaCl	78.00	275
Water (spring)	H_2O	.....	10 000

[a] Anhydrous: equivalent to 29.7 parts of commercial magnesium and sulphate, $MgSO_4.7H_2O$.

8. METHOD OF TESTING

In making a pull-out test, the specimen was placed on the upper head of the testing machine, as shown in Fig. 2, with the long end of the bar passing down through a central hole in the spherical bearing block. The long end of the bar was engaged in the wedge grips in the moving head of the machine. In a number of cases readings of load were lost, due to the slipping of the bar in the

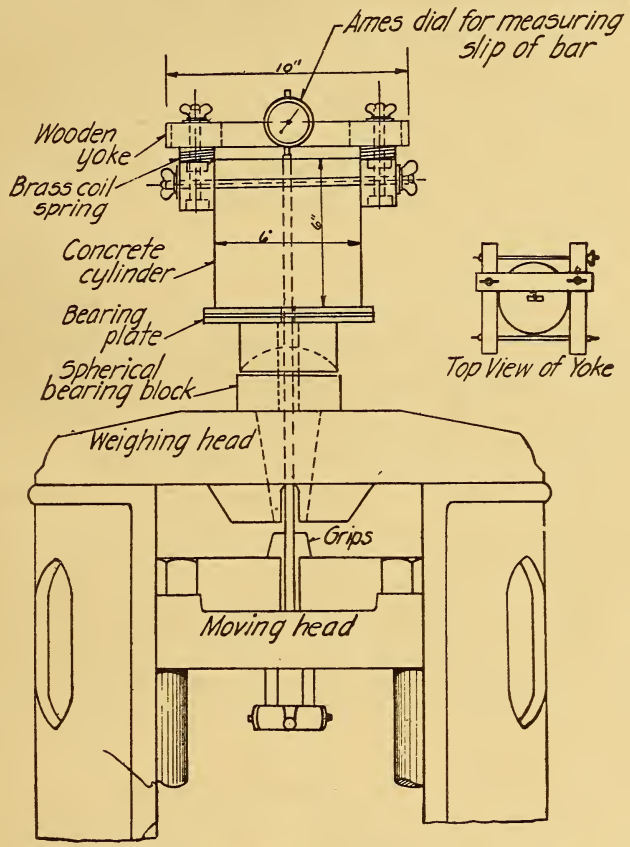


FIG. 2.—*Pull-out specimen in machine in position for testing*

grips at low loads, and in all cases this slipping made it difficult to keep the weighing beam of the machine in balance.[5]

The concrete block was embedded in plaster of Paris on a bearing plate, which transmitted the stress through a spherical bearing block to the weighing head of the machine. The plaster of Paris served to give a uniform bearing between the specimen and the

[5] Since it is particularly desirable to obtain accurate readings of load at the beginning of the slip of the bar, this test could probably be improved by providing a better connection between the bar and pulling head. This could be accomplished by threading the end of the bar and holding it with a nut against a plate bearing against the lower side of the pulling head.

bearing plate, and the spherical bearing allowed the bar to be adjusted in a vertical position, thus preventing the bending of the bar which otherwise would have occurred when the axis of the bar was not normal to the bearing surface of the concrete block.

The slip of the bar was measured at the upper end of the embedded bar by means of an Ames gage; this was supported by a wooden yoke clamped to the upper part of the concrete block in such a position that the plunger rested on the end of the bar. The dial was graduated so that its smallest division indicated a movement of 0.0001 inch, and a movement of 0.00005 inch was easily estimated. The latter was the smallest slip recorded. Load was applied continuously with the weighing beam kept carefully balanced, and as predetermined amounts of slip were reached, the corresponding load was read and recorded.

The tests were made on a power-driven Olsen testing machine of 20 000 pounds capacity. The slowest speed of the machine was used, giving the pulling head a movement of about 0.07 inch per minute. Due to the slipping of the bar in the grips at low loads, the stretching of the bar, and the compression in the plaster bearing, the slipping, as measured by the Ames gage, was much less than the movement of the pulling head.

9. NATURE OF THE BOND RESISTANCE OF A COATED BAR

A study of bond between concrete and uncoated steel has indicated that bond resistance is made up of two parts, adhesive resistance and sliding resistance. Adhesive resistance comes into play before the bar begins to slip. Sliding resistance is evidently due to friction between the concrete and the surface of the reinforcing bar, which may be, comparatively speaking, rough and irregular in form.

With the deformed bar the projecting lugs provide additional resistance after slipping has begun. It is believed that the adhesive resistance between the concrete and the longitudinal surfaces of the bar is destroyed before there is enough movement to develop much compressive stress between the beveled surface of the lug and the concrete in contact with it.

The action of a coated bar embedded in concrete is undoubtedly of a different nature from that of an uncoated bar. With the more plastic coatings especially, slipping may be due to three possible causes, as follows: (1) Slip between the coating and the concrete; (2) flowing or shearing deformation in the coating in the direction of motion; and (3) slip between the coating and the bar.

A very soft paint may act merely as a lubricant tending to facilitate slipping of the bar, while a hard, brittle paint may crumble when acted upon by shearing forces. Although the resistance of these paints to distortion is not known, it seems likely that the shearing strength of a paint is less than its adhesion to steel or concrete. On the other hand, a metallic coating, such as zinc, is elastic and has a comparatively high shearing strength, so that failure might be expected to occur through slipping of the zinc on the concrete. A study of the tests shows a considerable variation in the behavior of the different coatings.

10. GENERAL CHARACTERISTICS OF RESULTS

The bond-slip data of all tests have been plotted in Figs. 4 to 38, inclusive, which show the relation between bond stress and slip of bar. Each figure contains the information secured with the use of a certain coating.

The values of bond stress for end slips up to 0.01 inch have been plotted on an extended scale in the left-hand portion of each diagram in order to show distinctly the zone in which end-slip values began to increase rapidly; the same data are included in the right-hand portion of each diagram, which shows values of bond stress for end slips up to 0.20 inch. The points plotted in each diagram represent the average results of tests of two similar specimens. The different conditions of storage have been indicated by the same conventions throughout the diagrams.

The form of the bond-slip curves for coated bars is, in general, similar to that for uncoated bars. A few outstanding differences may be noted briefly. While the plain uncoated bar reached its maximum bond resistance at a slip of 0.01 inch or less, after which the bond stress decreased with further slip, many of the plain painted bars did not reach maximum bond resistance until a slip of 0.03 to 0.05 inch had occurred, and the bond stress remained nearly constant with considerable increase in slip.

However, with some metallic coatings the maximum bond resistance on plain bars was developed at a slip of 0.001 inch and was followed by a sudden large slip of bar with a correspondingly large reduction in bond stress.

Uncoated deformed bars showed an increase in bond resistance with an increase in slip throughout the test. The results for coated deformed bars show two noticeable features: (1) The bond stress was low for small amounts of slip, but the general slope of the bond-slip curves is about the same as for the uncoated bars; (2) in some cases the bond stress developed at the initial slip

remained constant, or even decreased, until a slip of 0.02 to 0.03 inch was reached, after which the bond-slip curves took on the characteristic slope of the curve for uncoated bars. It is evident that the point at which a curve takes this sudden change in slope corresponds to the slip at which the lugs of the deformed bar have reached a firm bearing against the adjoining concrete.

Tests reported elsewhere[6] have indicated that a very small amount of slip corresponds to critical bond stress in a reinforced concrete beam. For this reason comparisons have been made of bond stresses corresponding to a slip of 0.001 inch and 0.01 inch, respectively. The ratios of these stresses to the maximum bond stresses developed in these tests are given in Table 6 in the form of percentages.

TABLE 6.—Bond Stresses at Specified Slips in Percentage of Maximum Bond Stresses

[Values given are averages for all ages and conditions of storage]

Coating		0.001-inch slip		0.01-inch slip	
Reference letter	Kind	Plain bars	Deformed bars	Plain bars	Deformed bars
		Per cent	Per cent	Per cent	Per cent
A	(No coating used)	94	53	99	62
C	Asphaltic	58	8	91	13
D		60	23	90	36
F		55	15	79	22
G		53	23	86	33
H		74	33	95	44
M		62	26	92	45
	Average	60	21	89	32
B	Coal tar	65	20	88	34
N		68	26	98	37
	Average	67	23	93	36
E	Ferric oxide	67	24	86	38
I		84	33	97	44
K		66	.....	84	.....
L		73	29	95	41
	Average	73	29	90	41
O	Metallic	98	50	80	44
P		100	66	53	45
X		101	.....	74	.....
	Average	100	58	69	45
Q	Metal spray	16	4	76	12
J	Red lead	47	7	66	15
V	Phosphate	92	49	97	56

[6] Abrams, Bull. 71, Eng. Exp. Sta., Univ. of Illinois.

In order to present graphically a large amount of the information of these tests, Fig. 3 has been prepared. Bond stresses at slips of 0.001 inch and 0.01 inch, and at maximum load are shown. The chart shows that, in general, the bond resistance of coated bars was much less than that for uncoated bars. The dotted horizontal lines drawn in Fig. 3, corresponding to the bond stresses developed in uncoated bars at a slip of 0.001 inch and at the maximum load, enable this comparison to be made readily. Averaging the results for all conditions of storage for each coating, the data of Fig. 3 are summarized and shown as percentages in Table 7.

TABLE 7.—Bond Stresses for Coated Bars in Percentage of Bond Stresses for Uncoated Bars

[Values given are averages for all ages and conditions of storage]

Coating		Percentage of bond stress for uncoated bars						Average
Reference letter	Kind	0.001-inch slip		0.01-inch slip		Maximum bond stress		
		Plain	Deformed	Plain	Deformed	Plain	Deformed	
A	Uncoated	100	100	100	100	100	100	100
C	Asphaltic	30	9	43	14	47	69	.....
D		29	29	41	37	45	65	.....
F		30	19	40	23	50	67	.....
G		28	29	42	36	49	68	.....
H		38	45	46	50	48	71	.....
M		48	32	66	48	71	65	.....
	Average	34	27	46	35	52	67	43
B	Coal tar	53	25	67	36	76	67	.....
N		62	38	83	47	84	78	.....
	Average	58	32	75	42	80	73	60
E	Ferric oxide	30	35	36	49	42	80	.....
I		45	35	48	40	49	56	.....
K		37	.....	43	.....	52	.....	.....
L		40	38	48	47	50	71	.....
	Average	38	36	44	45	48	69	47
O	Metallic	76	78	58	59	72	82	.....
P		89	94	42	69	80	76	.....
X		97	.....	66	.....	89	.....	.....
	Average	87	86	55	64	80	79	75
Q	Metal spray	1	7	5	17	7	88	21
J	Red lead	6	9	8	16	8	63	18
V	Phosphate	90	107	88	104	91	116	99

11. ACTION OF INDIVIDUAL COATINGS

A brief tabular summary of the characteristic action of the various coatings is given in Table 8. As an additional guide in determining the merits of a given type of coating, the resistance of each coating to salt-spray corrosion tests is also presented.

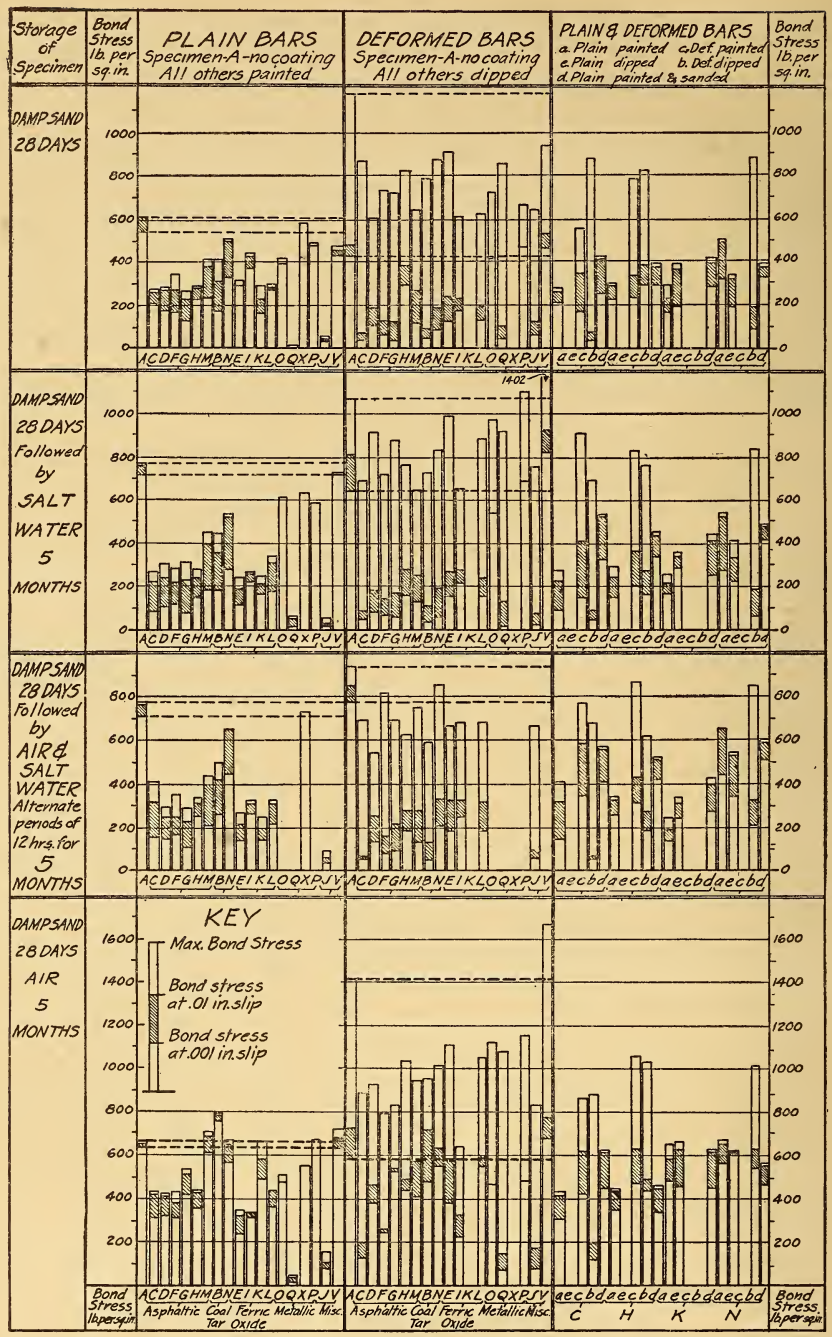


FIG. 3.—*Bond stresses developed with different coatings under various conditions of storage*

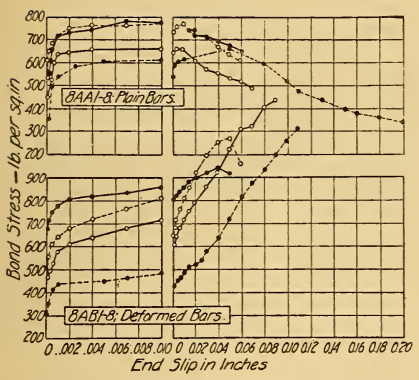


FIG. 4.—*Bond-slip diagrams for uncoated bars*

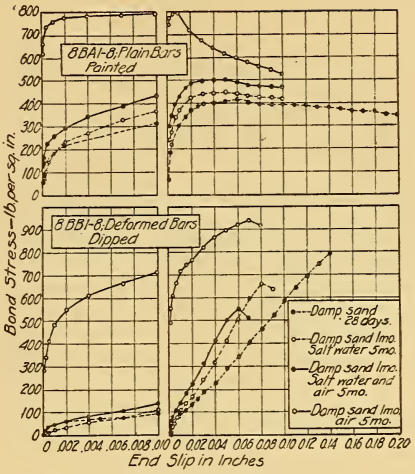


FIG. 5.—*Bond-slip diagrams for coating B; coal tar*

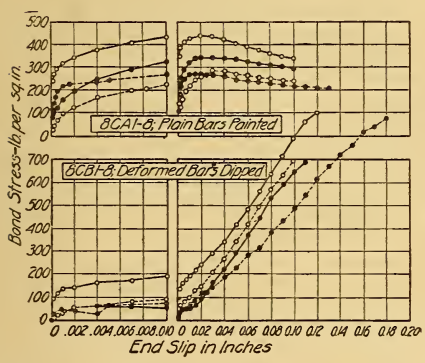


FIG. 6.—*Bond-slip diagrams for coating C; asphaltic*

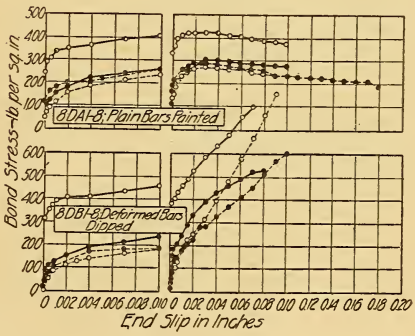


FIG. 7.—*Bond-slip diagrams for coating D; asphaltic*

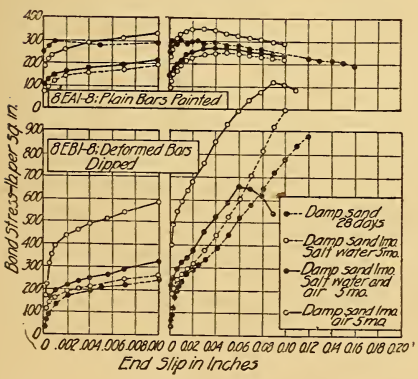


FIG. 8.—*Bond-slip diagrams for coating E; ferric oxide*

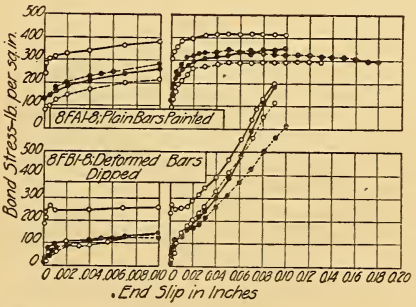


FIG. 9.—*Bond-slip diagrams for coating F; coal tar*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

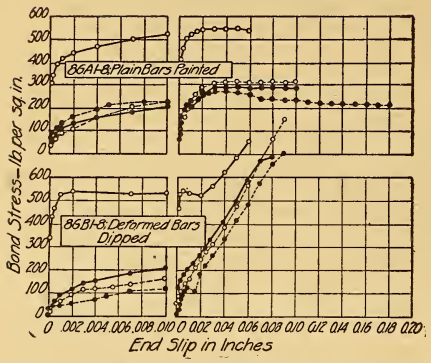


FIG. 10.—*Bond-slip diagrams for coating G; asphaltic*

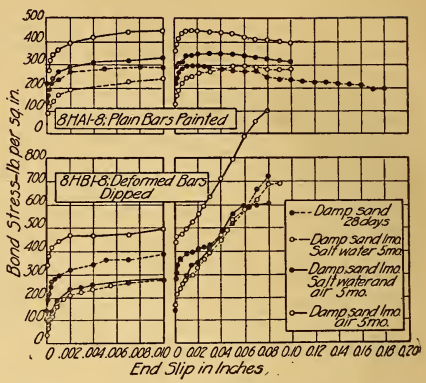


FIG. 11.—*Bond-slip diagrams for coating H; asphaltic*

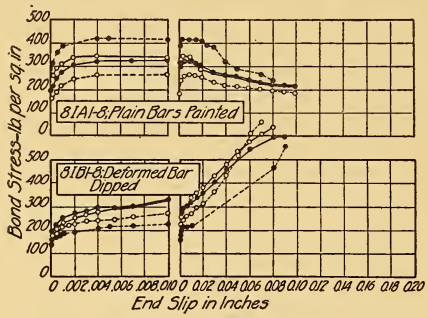


FIG. 12.—*Bond-slip diagrams for coating I; ferric oxide*

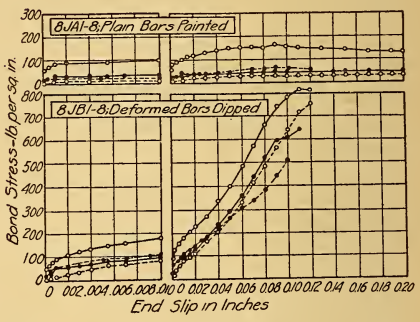


FIG. 13.—*Bond-slip diagrams for coating J; red lead*

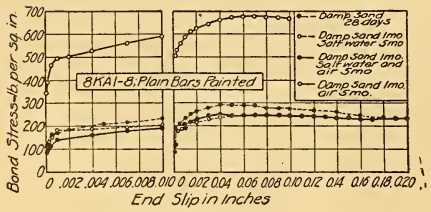


FIG. 14.—*Bond-slip diagrams for coating K; ferric oxide*

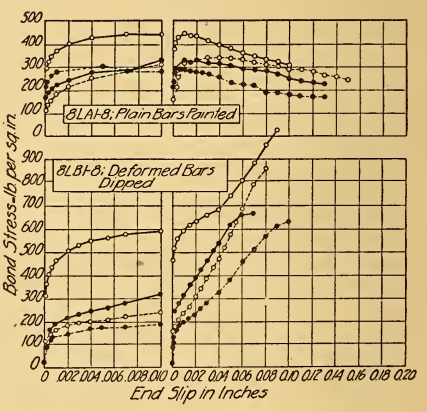


FIG. 15.—*Bond-slip diagrams for coating L; ferric oxide*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

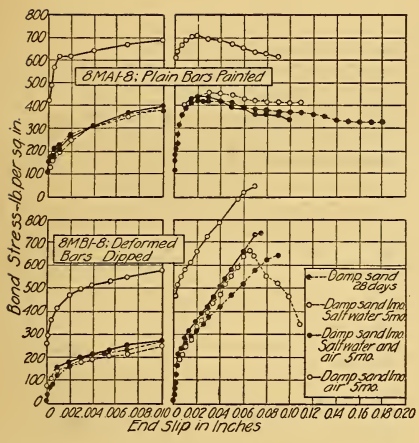


FIG. 16.—*Bond-slip diagrams for coating M; asphaltic*

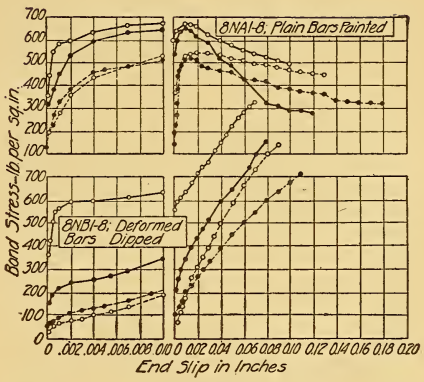


FIG. 17.—*Bond-slip diagrams for coating N; coal tar*

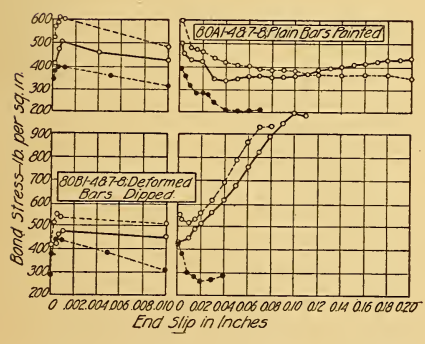


FIG. 18.—*Bond-slip diagrams for coating O; galvanized*

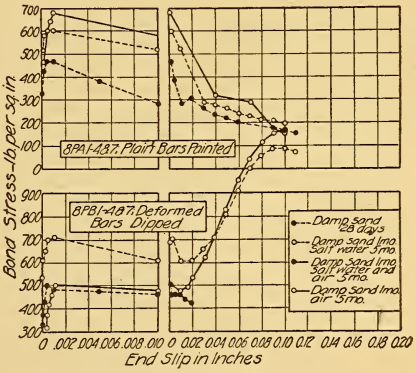


FIG. 19.—*Bond-slip diagrams for coating P; sherardized*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

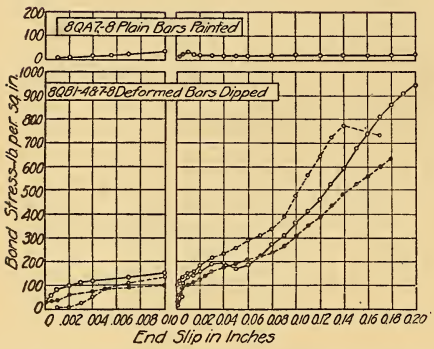


FIG. 20.—*Bond-slip diagrams for coating Q; metal spray*

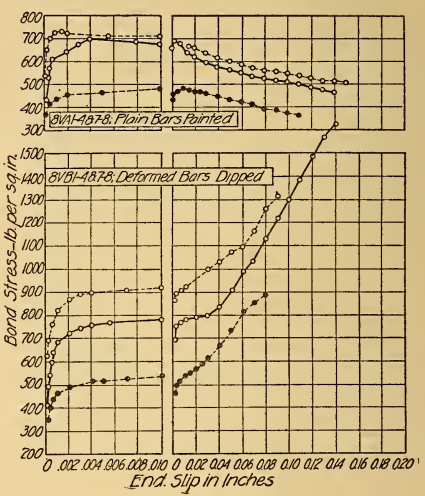


FIG. 21.—*Bond-slip diagrams for coating V; phosphate film*

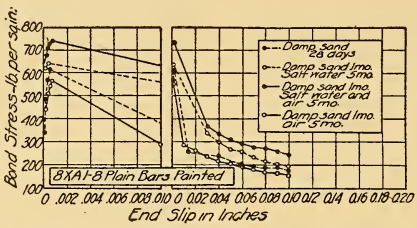


FIG. 22.—*Bond-slip diagrams for coating X; electro galvanized*

NOTE FOR FIGS. 4 to 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

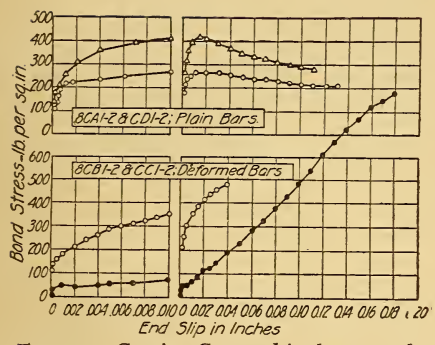


FIG. 23.—*Coating C, stored in damp sand 28 days*

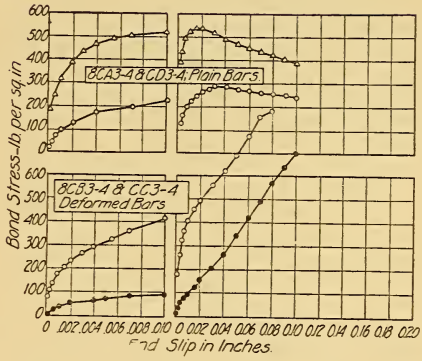


FIG. 24.—*Coating C, stored in damp sand one month, salt water five months*

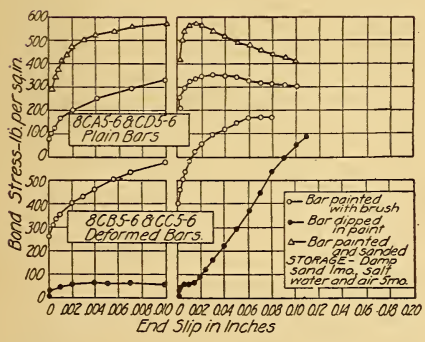


FIG. 25.—*Coating C, stored in damp sand one month, salt water and air five months*

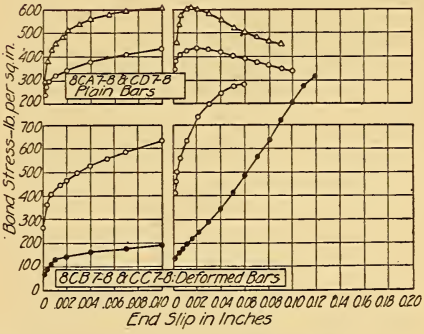


FIG. 26.—*Coating C, stored in damp sand one month, air five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

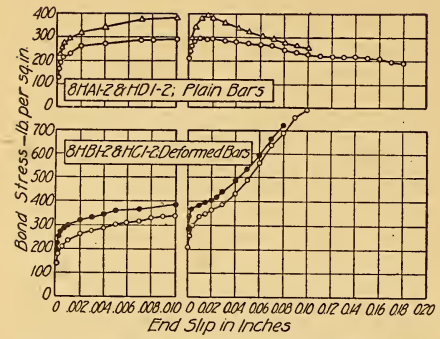


FIG. 27.—*Coating H, stored in damp sand 28 days*

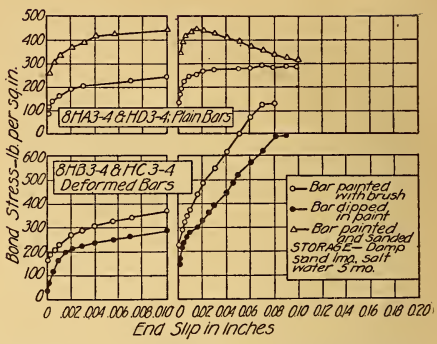


FIG. 28.—*Coating H, stored in damp sand one month, salt water five months*

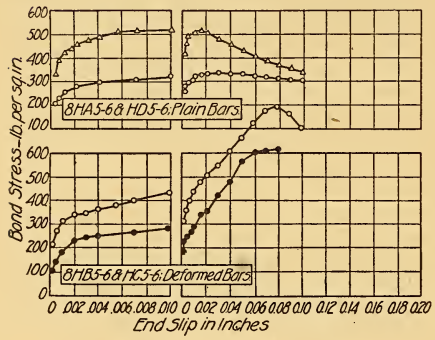


FIG. 29.—*Coating H, stored in damp sand one month, salt water and air five months*

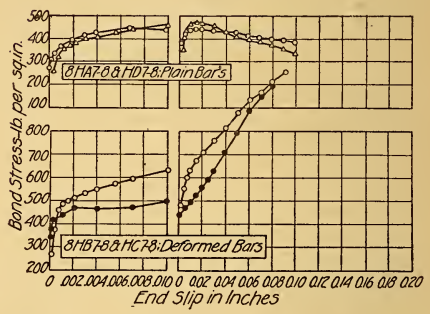


FIG. 30.—*Coating H, stored in damp sand one month, air five months*

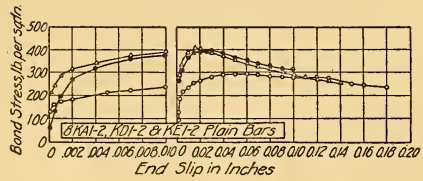


FIG. 31.—*Coating K, stored in damp sand 28 days*

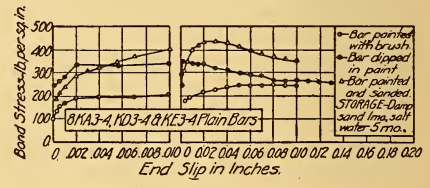


FIG. 32.—*Coating K, stored in damp sand one month, salt water five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

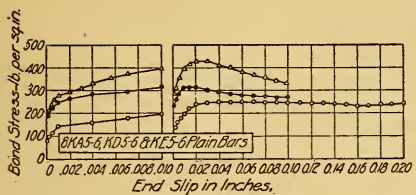


FIG. 33.—*Coating K, stored in damp sand one month, salt water and air five months*

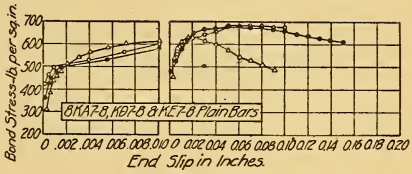


FIG. 34.—*Coating K, stored in damp sand one month, air five months*

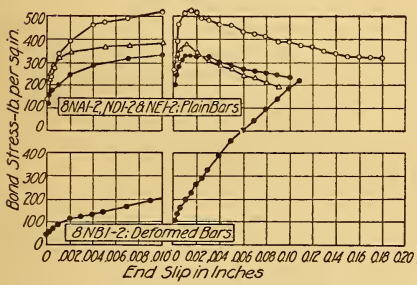


FIG. 35.—*Coating N, stored in damp sand 28 days*

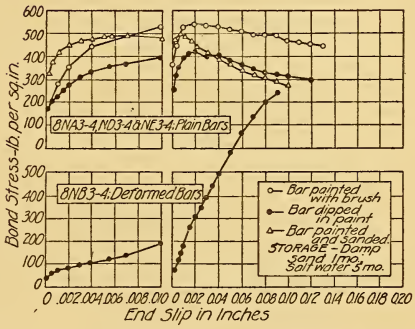


FIG. 36.—*Coating N, stored in damp sand one month, salt water five months*

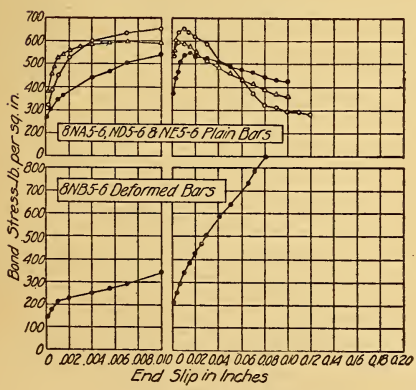


FIG. 37.—*Coating N, stored in damp sand one month, salt water and air five months*

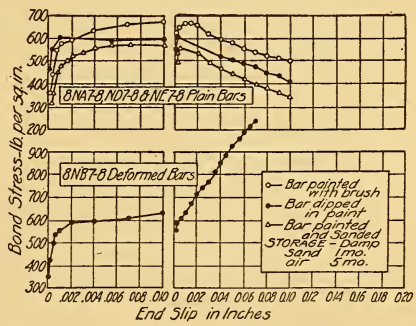


FIG. 38.—*Coating N, stored in damp sand one month, air five months*

NOTE FOR FIGS. 4 TO 38, INCLUSIVE.—In the left-hand section of each figure the slip, up to 0.01 inch, is plotted horizontally to a more extended scale than that which is used in the right-hand section.

TABLE 8.—Characteristic Action of Coatings in Bond Tests and Corrosion Tests

Reference letter	Composition and appearance of coating	Average time of drying		Ratio of bond stress for coated bar to uncoated bar (0.01-inch slip)		Characteristic feature of bond tests	Resistance to salt-spray corrosion test
		Plain	Deformed	Plain	Deformed		
		Days	Days				
B	Coal tar; dried to dull finish, with tough adhesive film.	2	5	0.67	0.36	Bond stress high after 6 months' storage in air; fairly low after other storage; above average on plain bars.	Failed.
C	Asphaltic; surface sticky and not very adhesive.	3	6	.43	.14	Comparatively low, especially on deformed bars; below average of all coatings.	Fair.
D	Asphaltic; fairly hard and tough surface.	3	5	.41	.37	Comparatively low; below average of all coatings.	Do.
E	Ferric oxide; adhesive surface.	2	6	.36	.49	Better result on deformed than on plain bars; about average of all coatings; shows well on 28-day test.	Good.
F	Coal tar; dried to dull finish with tough adhesive film.	2	5	.40	.23	Bond stress high after 6 months' storage in air; fairly low after other storage; above average on plain bars.	Fair.
G	Asphaltic; slightly sticky, glossy and adhesive.	2	6	.42	.36	Slightly below average of all coatings; best after 6 months' storage in air; maximum bond of plain bars continues with considerable slip.	Do.
H	Asphaltic; film sticky and glossy, not very adhesive.	2	6	.46	.50	Slightly above average; paint was scaly after specimen was broken and adhered partly to concrete.	Do.
I	Ferric oxide; hard, dull, rather rough finish.	3	3	.48	.40	Average of all coatings; adhered to concrete of broken specimens; form of bond-slip curves similar to those of uncoated bars.	Good.
J	Red lead; thick, wrinkled film; dried very slowly.	.....	.....	.08	.16	Very poor bond; fair maximum bond on deformed bars; paint adhered to both steel and concrete of broken specimen; appeared chalky.	Excellent.
K	Ferric oxide; smooth, glossy film.	.....	.....	.43	.....	Slightly below average; fair after 6 months' storage in air; maximum bond continued with large slips.	Do.

TABLE 8—Continued

Reference letter	Composition and appearance of coating	Average time of drying		Ratio of bond stress for coated bar to uncoated bar (0.01-inch slip)		Characteristic feature of bond tests	Resistance to salt-spray corrosion test.
		Plain	Deformed	Plain	Deformed		
		Days	Days				
L	Ferric oxide; paint thick and poor consistency.	5	3	0.48	0.47	Slightly above average; best results with 6 months' storage in air; adhered mainly to concrete in broken specimens.	Failed.
M	Asphaltic; hard, glossy film.	7	4	.66	.48	Better than average; paint appeared slightly brittle in broken specimens.	Fair.
N	Coal tar; film lumpy, not of uniform thickness.	7	4	.83	.47	Considerably above average; bond-slip curves similar to those for uncoated bars; paint adhered to broken specimens.	Failed.
O	Zinc (galvanized); film about 0.003 inch thick.	.....	.....	.58	.59	Much above average; maximum bond at 0.001-inch slip on plain bars followed by sudden slip and decrease of bond; peculiar bond-slip curves for deformed bars.	
P	Zinc (sherardized); film about 0.002 inch thick.	.....	.....	.42	.69	Much above average; maximum bond on plain bars at 0.001-inch slip; action similar to "O."	
Q	Zinc (metal spray); film about 0.001 inch thick.	.....	.....	.05	.17	Very poor bond; a fair maximum bond is developed with deformed bars; practically no bond with plain bars.	
V	Phosphate.....	.....	.....	.88	1.04	Very good bond; action similar to that of uncoated bars; consistently good at all storages.	
X	Zinc (electro galvanized)	.....	.....	.66	.....	Good bond, very much above average; behavior similar to that of coatings "O" and "P."	

12. COMPARISON OF PLAIN AND DEFORMED BARS

The relation between bond stresses developed by plain bars and those developed by deformed bars has already been shown in the bond-slip curves and in Fig. 3. In order to provide a definite numerical comparison, the information of Table 9 was prepared. Results given for coated bars are the average for all coatings tested.

TABLE 9.—Comparison of Bond Stresses in Plain and Deformed Bars

[Values given are averages for all ages and all conditions of storage]

Treatment of bar	Bond stress at 0.001-inch slip		Ratio, plain to deformed	Bond stress at 0.01-inch slip		Ratio, plain to deformed	Maximum bond stress		Ratio, plain to deformed
	Plain	Deformed		Plain	Deformed		Plain	Deformed	
	Lbs/in.²	Lbs/in.²		Lbs/in.²	Lbs/in.²		Lbs/in.²	Lbs/in.²	
Uncoated	655	610	1.07	704	715	0.99	708	1153	0.62
Coated	288	220	1.31	348	293	1.19	400	836	.48

Conclusions based on the relation of bond stresses for plain and deformed bars, as shown in Table 9, must be qualified by the fact that the method of applying the coatings was different for the plain bars from that used for the deformed bars. This subject is discussed in Section II–14.

13. EFFECT OF VARIATIONS IN CONDITIONS OF STORAGE

Fig. 3 shows in a general way that the bond strength for specimens six months old was considerably higher for those stored in air than for the corresponding specimens stored under other conditions. In order to facilitate comparison, Table 10 has been prepared. This table shows the effect on the bond strength of the various conditions of storage.

With plain bars continuous storage in sea water produced about the same effect as alternate storage in sea water and air. Bond stresses from both of these averaged about three-fourths as high as those developed with the specimens stored in air. The effect of a given method of storage does not show consistent results for the plain and deformed types of bar, but in general no appreciable difference in bond stress may be expected between the two conditions of continuous and intermittent exposure to sea water.

The unprotected ends of bars were much more badly corroded in the case of intermittent exposure to sea water than in the case

of continuous exposure, but examination of the specimens showed that this condition did not extend to the embedded portion of the bars.

TABLE 10.—Effect of Conditions of Storage on Bond Strength

[Maximum bond stress for various methods of storage in percentage of maximum bond stress for storage in damp sand 28 days and in air 5 months]

Coating		Plain bars			Deformed bars		
Reference letter	Kind.	Damp sand 28 days	Damp sand 28 days Sea water 5 months	Damp sand 28 days Air and sea water 5 months	Damp sand 28 days	Damp sand 28 days Sea water 5 months	Damp sand 28 days Air and sea water 5 months
		Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
A	Uncoated	92	116	116	83	75	66
C	Asphaltic	64	61	79	97	78	77
D		67	64	69	65	98	58
F		80	68	83	92	91	101
G		50	58	54	87	105	83
H		65	63	77	79	74	59
M		59	64	63	68	69	78
	Average	64	63	71	81	86	76
B	Coal tar	51	57	64	83	76	61
N		76	81	97	87	82	84
	Average	64	69	81	85	79	73
E	Ferric oxide	84	74	77	81	89	60
I		128	77	95	95	102	105
K		43	37	37	.....	.....	.....
L		68	78	75	59	84	64
	Average	81	66	71	78	92	76
O	Metallic	81	120	.....	63	87	.....
P		71	89	.....	43	95	.....
X		105	113	129	.....	.....	.....
	Average	86	107	129	53	91	.....
Q	Metal spray	.....	125	.....	72	84	.....
J	Red lead	35	36	56	77	92	80
V	Phosphate	67	103	.....	57	84	.....
	Average (all specimens)	71	78	78	76	86	75

14. EFFECT OF MANNER OF APPLYING COATINGS

In order to determine whether or not the bond strength of coated bars was affected by the manner of applying the coatings, specimens were made with four paints, C, H, K, and N (see Table 2), using three methods of application. These paints represented the asphaltic, ferric oxide, and coal-tar types. The bond-slip curves for this particular study are plotted in Figs. 23 to 38, and the

information is also shown in Fig. 3. The curves show some contradictory results, in that for cases which are comparable the highest bond resistance was sometimes obtained with painted bars and sometimes with dipped bars.

Table 11 shows relative values of bond stresses at 0.01 inch slip for all methods of applying coatings. For convenience, results from painted bars have been taken as unity. As four paints only were subjected to this comparison, the relations shown in this table should not be regarded as conclusive.

TABLE 11.—Relative Values of Bond Resistance at 0.01-Inch Slip—Effect of Various Methods of Applying Coating

Method of applying coating	"C" asphaltic		"H" asphaltic		"K" ferric oxide	"N" coal tar
	Plain bar	Deformed bar	Plain bar	Deformed bar	Plain bar	Plain bar
Painting	1.00	1.00	1.00	1.00	1.00	1.00
Dipping	.....	0.19	.....	0.84	1.46	0.70
Painting and sanding	1.74	.....	1.45	.....	1.65	.86

15. SUMMARY

Results of this series of tests have shown a wide variation in the behavior of different coatings. A few of the results may be summarized here for convenient reference.

(*a*) The maximum bond stress developed by bars which had been painted was generally considerably less than for unpainted bars, but the reduction in maximum bond stress due to galvanizing and some similar processes was less than that due to painting.

(*b*) With the exception of certain metallic coatings, the maximum bond stress for plain coated bars was reached after considerably greater slip than with uncoated bars. The amount of slip of coated bars at maximum stress was often 0.02 to 0.03 inch. Furthermore, the bond resistance did not decrease as rapidly as with uncoated bars with continued slip after the maximum load had been reached. Certain metallic coatings, however, reached maximum bond resistance with very small slip, followed by a sudden decrease in bond load with increasing slip. In a reinforced concrete beam under a constant load, such a yielding of the bond resistance might cause failure without warning.

(*c*) Coated deformed bars apparently slipped considerably before the corrugations or lugs reached a firm bearing. After this had taken place, the increase of resistance with increase of slip was similar to that in uncoated deformed bars.

(*d*) The bond resistance at a slip of 0.001 inch was a much smaller proportion of the maximum bond resistance for coated bars than for uncoated bars. A slip of 0.001 inch has been considered by some[7] to produce critical conditions of bond stress in a beam.

(*e*) For pull-out specimens employing coated bars, continuous and intermittent storage in artificial sea water resulted in bond strength averaging about three-fourths as great as from storage in air.

(*f*) Sanding of painted bars tended to increase the bond resistance in a number of cases; also dipping bars in paint often resulted in a lower maximum bond stress than was found for bars painted with a brush. However, the effectiveness of the manner in which a coating is applied seemed to depend upon the nature of the paint, and should be determined by test in every case in which it is important to have this information.

III. TESTS OF REINFORCED CONCRETE BEAMS WITH TENSION BARS LAPPED IN A REGION OF UNIFORM BENDING MOMENT

1. PRELIMINARY

This part of the investigation was made on 4 rectangular beams under two-point loading. Each beam had a one-half-inch reinforcing bar spliced in the region of uniform bending moment. The length of the lap was in all cases 41 inches (82 diameters) to insure a safe splice. The object in making tests with a lap of this amount was to secure data as a basis for recommendation as to the length of lap necessary (1) when continuous and spliced bars are adjacent, and (2) when all bars are spliced at the same section. There are a number of other conditions which should be studied in this kind of investigation, but it was not feasible to make these studies in connection with this work.

2. DESCRIPTION OF SPECIMENS

Four beams, 10 feet 8 inches long, 10 inches wide, and 12 inches deep, were made. In 3 of the beams the tension reinforcement consisted of 6 plain round bars, extending the full length of the beam (here termed through bars), and 2 short bars (here termed lapped bars), extending from opposite ends of the beam to a point 20½ inches beyond the center of the span. This gave the short bars a lap of 41 inches, which was symmetrical about the center line of the span. In the fourth beam no through

[7] Abrams, Bul. 71, Eng. Exp. Sta., Univ. of Illinois.

bars were present, but the lapped bars were placed in the same relative positions as in the other 3 beams. All lapped bars were one-half inch plain round. All bars were anchored at the ends of the beams by means of semicircular hooks. The lapped bars had no mechanical anchorage at the ends of the laps. The numbers and sizes of bars and the percentage of reinforcement in the several beams are summarized in Table 12.

TABLE 12.—Reinforcement of Beams with Lapped Bars

Beam No.	Through bars		Lapped bars		Reinforcement	Ratio [a] area of through bars to lapped bars
	Number	Diameter	Number	Diameter		
		Inch		Inch	Per cent	
8AAA1	0	.....	2	½	0.2	0
8AAB1	6	½	2	½	1.4	6
8AAC1	6	⅝	2	½	2.1	10.3
8AAD1	6	¾	2	½	2.9	13.5

[a] Only one lapped bar considered effective for determining this ratio.

For convenience in designating gage lines, the bars were numbered consecutively from 1 to 8, beginning with the bar next to

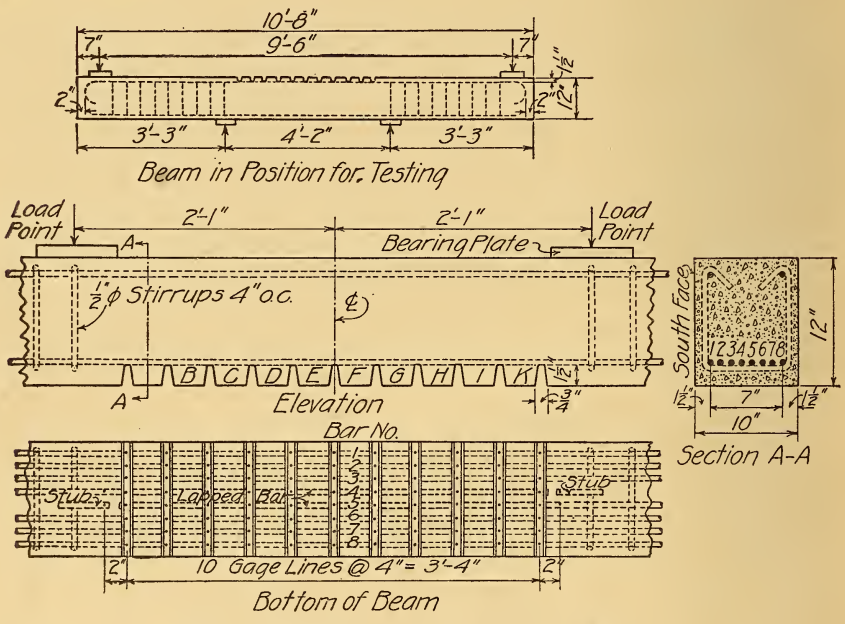


FIG. 39.—*Details of test beams with lapped bars*

the south side of the beam as placed in the testing machine, except for the beam having only the lapped bars. In this case the bars were numbered 4 and 5, the numbers given corresponding

to the numbers of the lapped bars in all the other beams. Fig. 39 shows the arrangement of the reinforcing bars in each beam. To prevent failure by diagonal tension, nine stirrups, made from one-half-inch plain round bars, were placed near both ends of each beam. Exposure at spots on the tension bars, for the purpose of taking strain gage readings, was provided for by placing wooden blocks in the bottom of the forms across the beams at each row of gage points before pouring. These blocks were about three-fourths inch wide and 1½ inches high and extended the full width of the beam. They were tapered slightly to facilitate removal.

The reinforcing steel used was of the quality known as shell discard. Average results of physical tests are given in Table 13.

TABLE 13.—Physical Properties of Reinforcing Steel Used in the Beam Tests

Diameter	Yield point	Ultimate tensile strength	Elongation in 8 inches	Reduction of area
	Lbs./in.²	Lbs./in.²	Per cent	Per cent
One-half inch	64 000	105 000	16	27
Five-eighths inch	50 000	90 000	21	46
Three-fourths inch	54 000	93 000	20	21

3. MATERIALS

The concrete was mixed in the approximate proportions of 1:1:1, by volume. The quantities of the materials were determined by weight. A rich mix was chosen in order to make the conditions closely similar in this respect to those in concrete ship construction. The fine aggregate was of two sizes: (1) A bank sand, all of which passed a one-eighth-inch screen; and (2) screenings from the gravel varying from one-eighth to one-fourth inch. The coarse aggregate consisted of pebbles from one-fourth to one-half inch in size. Lehigh Portland cement was used, samples of which were tested at the Bureau of Standards, Washington, D. C., and found to conform to the United States Government specifications for Portland cement.[8]

4. MAKING OF SPECIMENS

The blocks of wood used to expose the steel for strain gage readings were tacked to the bottom of the form, and the tension bars were fastened to them. A steel stub, one-half inch in diameter and 4½ inches long, was placed at the unanchored end of each

[8] Circular of the Bureau of Standards, No. 33.

lapped bar with the intention of taking readings with the strain gage between the stub and the end of the lapped bar. The stirrups were securely wired to the tension reinforcement, and were held in place at the top of the beam by wiring them to a one-half inch plain round bar which extended the full length of the beam. The same form was used for all four beams.

The concrete was mixed in a "Wonder" batch mixer of about 3 cubic feet capacity. Sufficient aggregate for each beam was taken from the storage bin, thoroughly mixed, and its moisture content was determined. Enough water was added to bring the total water content to 13 per cent of the total dry weight of the batch. Each batch was mixed for five minutes at the rate of 12 revolutions per minute, dumped into a tight box, and carried by a traveling crane to the form. As the concrete was shoveled into the form, it was puddled, and the form was jarred with hand hammers. Six 6 by 12 inch cylinders were made with each beam, 3 to be tested at the age of 7 days, and 3 at the same age as the beam when tested. Results of the tests of these specimens are shown in Table 14.

TABLE 14.—Strength of Beams and Control Cylinders

Beam No.	Maximum load [a]	Cylinder strength	
		7 days	27 days
	Pounds	Lbs./in.^{2}	Lbs./in.^{2}
8AAA1	5 600	3195	4830
8AAB1	55 150	3355	5465
8AAC1	51 300	3050	5215
8AAD1	76 400	3040	4575

[a] Beams 28 days old when tested.

The beams were removed from the forms 24 hours after pouring and were covered with wet burlap, which was sprinkled each day until the beams were prepared for the test.

5. METHOD OF TESTING

The tests were made in a 300 000-pound Olsen testing machine. Before placing the specimen in the testing machine, holes were drilled in the reinforcing bars for strain gage readings. The sides and bottom of the beam were whitewashed to facilitate the detection of cracks.

At each gage hole on the lapped bars a spot of whitewash was placed on the exposed bar and on the concrete immediately adjoining it to aid in the detection of slip at the lap during the

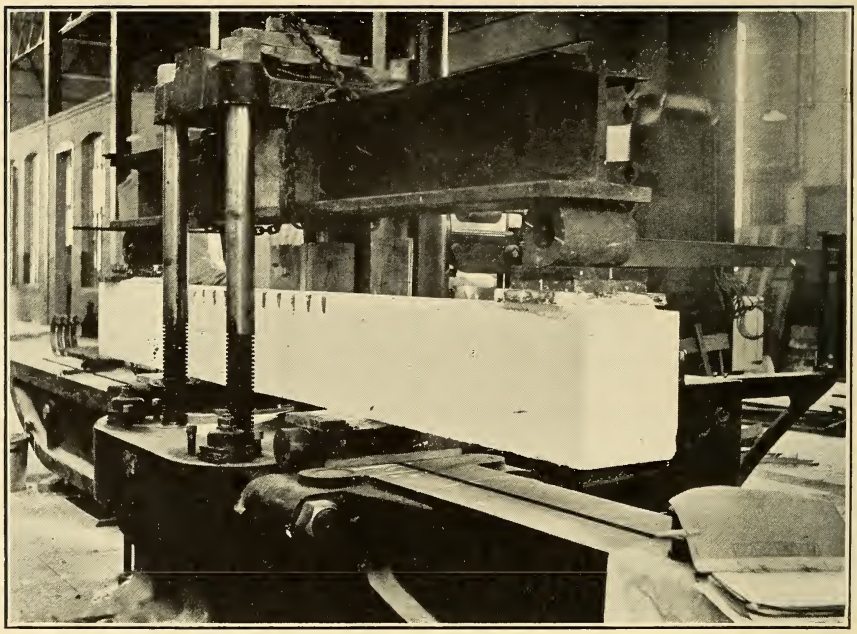


FIG. 40.—*Beam in testing machine ready for application of load*

test. The same procedure was followed on the through bars at the gage holes nearest the load points. It was found that these whitewashed spots gave a sensitive means of detecting any slip, as a minute movement was unfailingly shown by a crack in the whitewash on the sides of the spot parallel to the direction of slipping.

The beams were placed in the testing machine with the tension side uppermost, and were supported on the base of the machine at two points, each of which was 2 feet 1 inch from the center of the beam. They were loaded at the ends of the span through the medium of an H beam attached to the moving head of the testing machine. Fig. 40 is a view of one of the beams in the testing machine.

Since the points of support in these tests corresponded to the load points of beams tested in the normal position, they will be referred to in this paper as load points. The span was 9 feet 6 inches. Deflections were measured at the center of the beam. Readings of deformation were taken with a Berry strain gage on each of the bars in a series of 10 consecutive gage lines, each 4 inches long, 5 gage lines being taken on each side of the center line of the span. For each gage line on a lapped bar there was a gage line beside it on each through bar.

Sections through a beam bisecting a gage line on each bar were designated by letters from A to K, inclusive, except that letter J was omitted. Any gage line is identified by its section letter and bar number; thus K6 indicates the gage line which is cut by section K of bar No. 6. Bar No. 5 ended near section A, and bar No. 4 near section K, beyond the lettered section in each case.

Three sets of strain gage readings were taken at low loads on each beam to furnish a double check on the accuracy of the zero readings. As the test proceeded, strain gage readings were taken at various increments of load; length and direction of cracks were observed with corresponding loads; and the whitewashed observation points were watched to detect any slip of the lapped bars. As the strain gage readings were taken on both sections of the spliced bar at the lap, they are treated as two bars when discussing the phenomena of the lap, but considered as only one bar effective as tension reinforcement.

6. PHENOMENA OF THE TESTS

(*a*) Beam 8AAA1, poured on November 22, 1918, was set up for testing on December 20. This beam had as tension reinforcement two short one-half-inch plain round bars, lapped 41 inches

at the center of the beam, and no through bars. These bars were given the numbers 4 and 5 since they corresponded in positions with bars 4 and 5 of the other beams. Strain gage readings were taken at applied loads of 1000, 2000, 3000, 4000, 5000, and 5600 pounds.

Further details: No cracks appeared until a load of 3000 pounds had been applied. At this load a small tension crack 3½ inches long opened up in the northwest quarter, 20¼ inches from the center of the beam. A similar crack in the same relative position showed in the northeast quarter of the beam. A small crack about 1 inch long showed at the center of the beam. Cracking on the south face of the beam was similar to that on the north face. These cracks started from the depressions left in the concrete by the wooden blocks used to expose the reinforcing bars at the gage holes.

At a load of 4000 pounds the crack in the northwest quarter increased slightly in length. On the southeast face several very fine cracks opened up, showing no connection with each other and no tendency to take the same general direction. Each was about 1½ inches long.

At a load of 5000 pounds the crack in the northwest quarter had extended about 10¼ inches into the beam and had increased very much in width. It showed in similar manner in the southwest quarter. A small crack about 4¼ inches long opened up on the northeast face. After about one hour the load, which had fallen off slightly, was brought up to 5000 pounds, and another set of strain gage readings was taken. No new cracks had appeared. Upon increasing the load to 5300 pounds the tension crack at the east load point opened suddenly, and the load fell off slightly with the machine running; but it immediately picked up and increased to 5600 pounds, which was the maximum load attained. The machine was kept running for some time after the maximum load was reached, the beam holding up a load of 3900 pounds.

After removing the beam from the testing machine, an examination showed that bar No. 4 had slipped 0.26 inch at the unanchored end. On this bar all the whitewashed points showed the bond broken between steel and concrete and also indicated considerable movement of the bar. No. 4 showed movement at all of the observation points except the two nearest the unanchored end. These points were three-fourths inch and 4¾ inches, respectively, from the unanchored end. All of the other whitewashed observation points showed a disturbance of the bond by fine cracks in the whitewash parallel to the bar, but whatever movement there was in the bar was too small to measure by any means available.

(*b*) Beam 8AAB1, poured on December 3, was tested on December 31. The tension reinforcement of this beam consisted of six one-half-inch plain round through bars and two one-half-inch plain round short bars lapped 41 inches at the center of the beam. Strain gage readings were taken at loads of 1000, 3000, 5000, 8000, 22 000, 36 000, 50 000, and 55 150 pounds.

Further details: At a 5000-pound load two very fine cracks showed in the southwest quarter and one in the northwest quarter. These cracks were short tension cracks outside of the load points.

At an 8000-pound load small tension cracks appeared in all the quarters, none of which were over 3 inches long. Very fine cracks appeared at the ends of the lapped bars, also at the center of the beam.

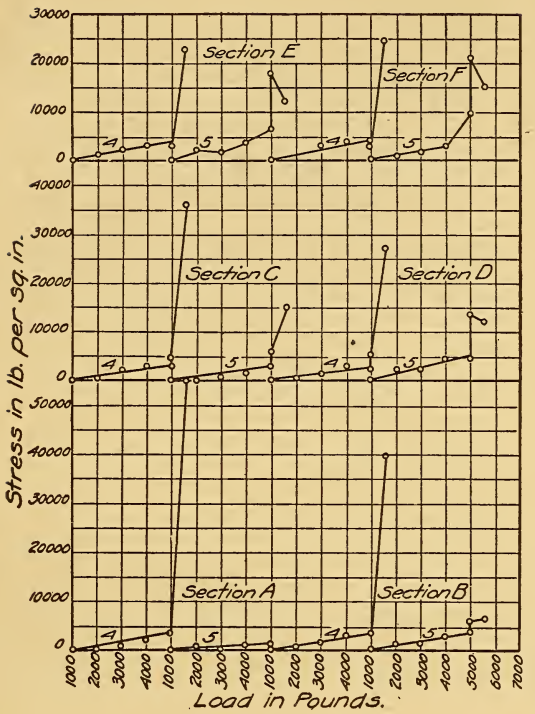


FIG. 41.—*Beam 8AAA1; lapped bars only. Sections A–F*

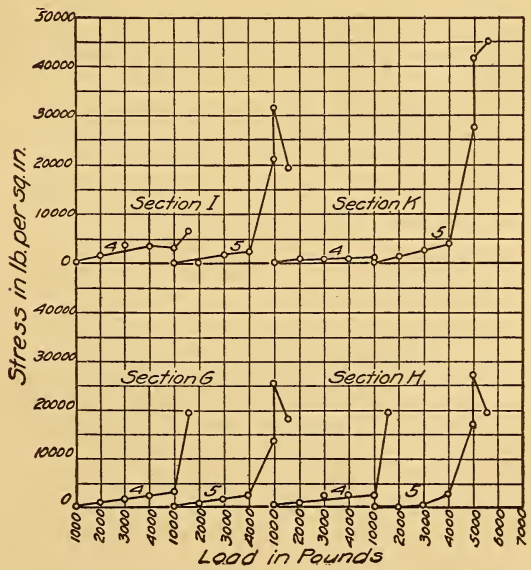


FIG. 42.—*Beam 8AAA1; lapped bars only. Sections G–K*

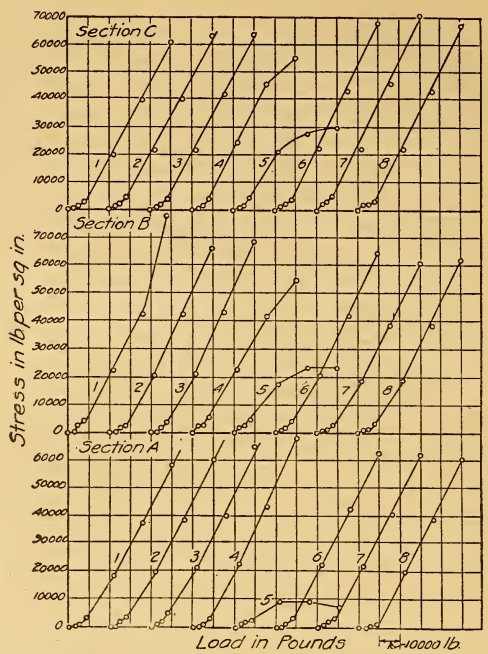


FIG. 43.—*Beam 8AABI; one-half inch through and one-half inch lapped bars. Sections A–C*

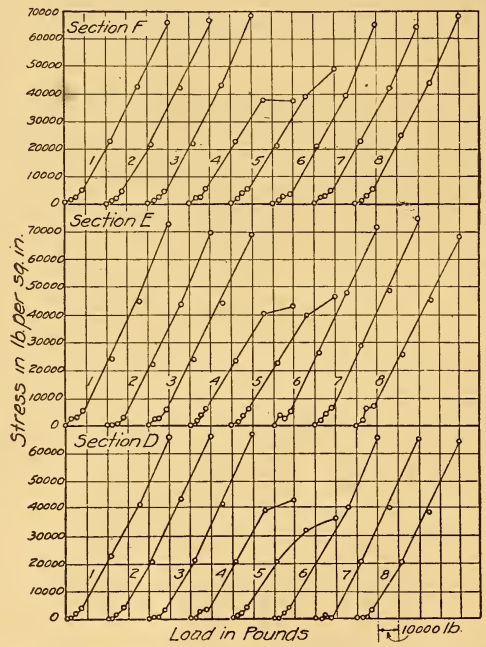


FIG. 44.—*Beam 8AABI; one-half inch through and one-half inch lapped bars. Sections D–F*

At a 22 000-pound load cracks opened up uniformly in all quarters and extended up into the beam about 7 inches. Between the load points tension cracks were quite uniformly spaced and extended into the beams about 6½ inches. No disturbance of the bond of the lapped bars could be seen at any of the observation points.

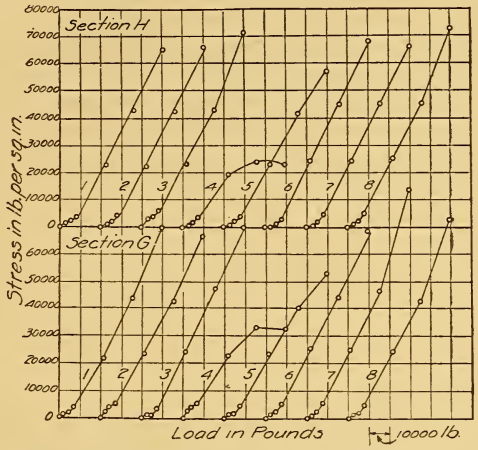


FIG. 45.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections G–H*

At a load of 36 000 pounds all the cracks had lengthened an appreciable amount, but showed no great increase in width. Bar No. 4 showed the bond broken at the first two observation points on the unanchored end at readings K4 and I4. None of the other points showed any movement.

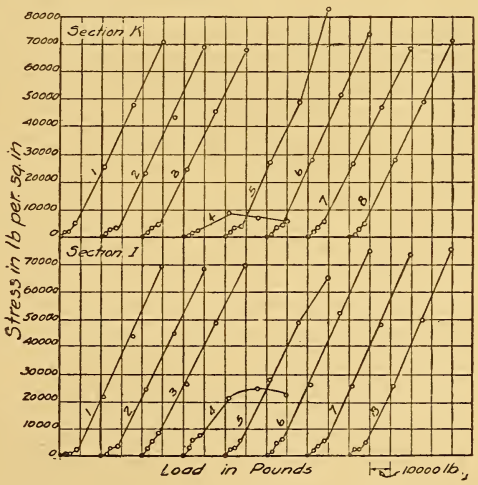


FIG. 46.—*Beam 8AAB1; one-half inch through and one-half inch lapped bars. Sections I–K*

At a 50 000-pound load the observation points on No. 4 lapped bar showed movement from the unanchored end to 4 inches beyond the center of the beam. On No. 5 bar the observation points from the unanchored end to the center of the beam showed movement. All of the cracks had increased in width, particularly the one

at the east end of the lapped bar. The cracks outside of the load points had the direction of diagonal tension cracks.

The load increased to 55 150 pounds and then dropped slightly, and it was evident from the width of the tension cracks that the yield point of the steel had been reached. Strain gage readings were taken. The No. 4 bar showed slip from the unanchored end to 12 inches beyond the center of the beam. Bar No. 5 showed movement at all the observation points to the center of the beam. The load continued at about 54 000 pounds for some time, falling off very slowly with the machine running. The test was discontinued after the center deflection had reached 1.35 inches. The observation points on bar No. 5 from the unanchored end to 12 inches beyond the center of the beam and those on bar No. 4 from the unanchored end to 16 inches beyond the center of the beam showed movement. No slip of the through bars was observed.

(*c*) Beam 8AAC1, poured on November 29, was made ready for test December 27. This beam had six five-eighths-inch plain round through bars and two short sections of one-half-inch plain round bars lapped at the center of the beam for tension reinforcement. Strain gage readings were taken at loads of 1000, 3000, 5000, 10 000, 30 000, 50 000, and 51 000 pounds.

Further details: At a load of 3000 pounds one small crack developed in the southeast quarter at the end of the lapped bar.

At the 5000-pound load a similar crack showed in the northeast quarter.

At the 10 000-pound load a small crack in the southeast quarter extended about 3 inches parallel to the horizontal steel. Small tension cracks showed in all quarters between the load points and the supports.

At the load of 30 000 pounds several cracks appeared between the load points extending about halfway up the sides of the beam. Outside of the load points there were two or three cracks in each quarter, those in the northwest quarter having the direction of diagonal tension cracks. None of the observation points, consisting of patches of whitewash on the bars, showed disturbance of bond. The crack in the southeast quarter parallel to the horizontal bars showed a slight extension, and most of the cracks in all of the quarters showed branches parallel to the horizontal bars.

At a 50 000-pound load diagonal tension cracks showed in all the quarters. The cracks between the load points had opened wider and extended deeper into the beam. Bar No. 5 (lapped) showed slip at all the observation points from the free end to 8 inches beyond the center of the beam. Bar No. 4 showed movement at all the points from the free end to the center of the beam. The outside through bar on the north side showed slip at the observation point nearest the west load point. Horizontal cracks along the outside bars between load points became somewhat longer. Also outside of load points horizontal cracks extended toward the ends of the beams.

On starting the testing machine the load increased to 51 300 pounds, then dropped to 51 000 pounds. The size of the cracks showed that the yield point of the steel had been reached. A set of strain gage readings was taken. The first observation point at the unanchored end of bar No. 4 was the only one unbroken on the lapped bars. The whitewashed observation points were broken on the two outside bars at readings A7 and A8, also at K1, K2, K3, K5, and K8, showing the slip of these bars.

Loading was continued, and cracks opened up uniformly between the load points. The load continued at about 50 000 pounds for some time. After the center deflection had reached 0.87 inch the test was discontinued. The load at this point was 49 700 pounds.

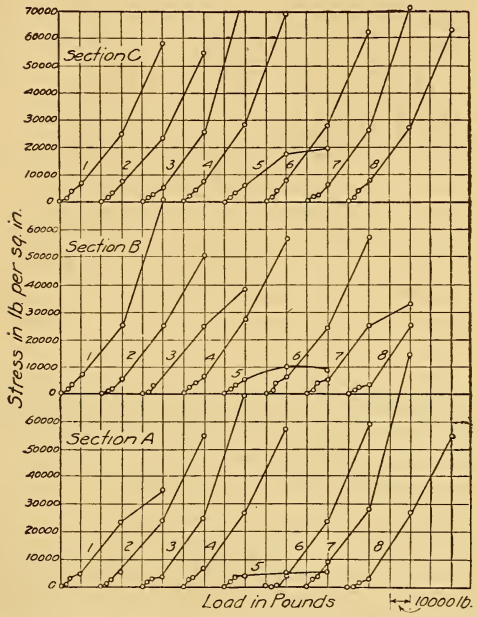


FIG. 47.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections A-C*

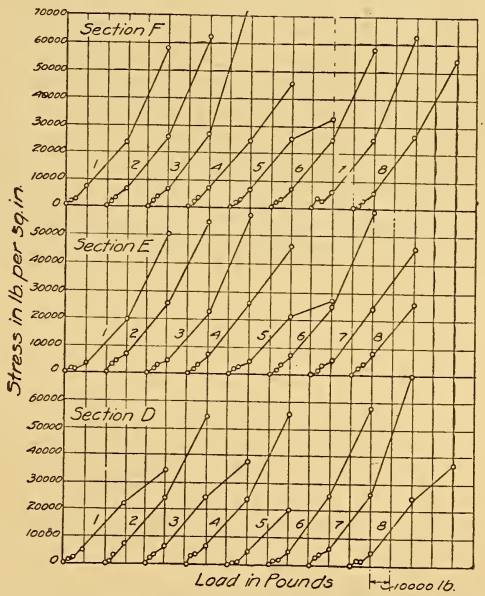


FIG. 48.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections D-F*

(*d*) Beam 8AAD1, poured on November 26, was set up for test December 24. This beam had for tension reinforcement six three-fourths-inch plain round through bars, anchored at both

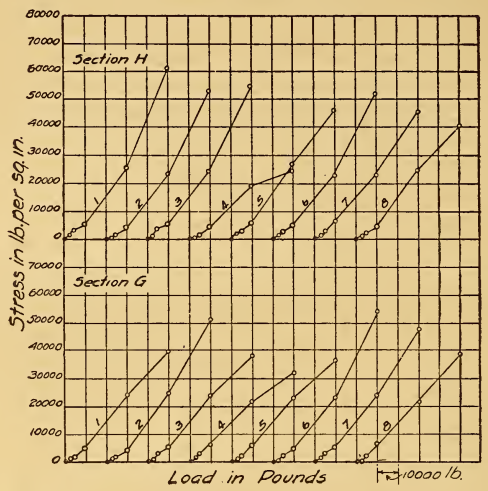


FIG. 49.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections G–H*

ends of the beam, and two short sections of one-half-inch plain round bars lapped at the center of the beam. On removing the wooden blocks to drill the reinforcing bars for strain gage readings,

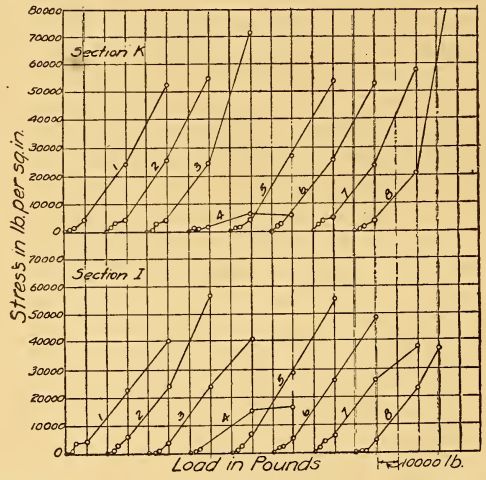


FIG. 50.—*Beam 8AAC1; five-eighths inch through and one-half inch lapped bars. Sections I–K*

the concrete around the bars was found to be porous. Fig. 51 shows a view of the bottom of this beam. All the concrete between the blocks was removed and a rich mortar carefully placed around the bars and rounded off above them. This was done a

FIG. 51.—*Porous condition around reinforcing bars of beam 8AAD1*

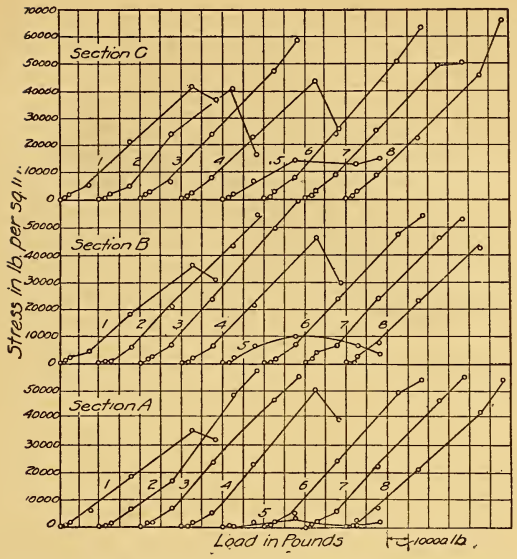


FIG. 52.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections A–C*

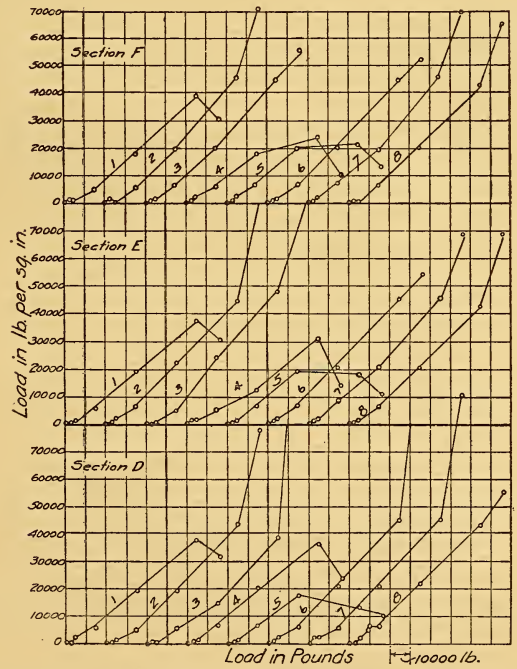


FIG. 53.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections D–F*

week before the test. Strain gage readings were taken at loads of 1000, 3000, 5000, 15 000, 35 000, 65 000, 76 400, and 73 000 pounds.

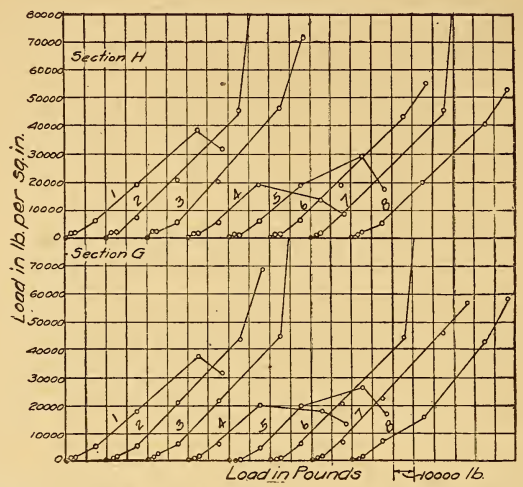


FIG. 54.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections G–H.*

Further details: At a load of 3000 pounds fine cracks showed in the sections at the free ends of the lapped bars.

At a load of 5000 pounds more fine cracks showed, but except for one on the southeast face, none were over 2 inches long.

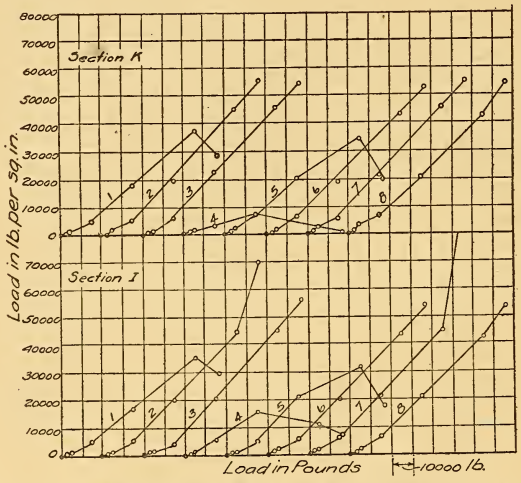


FIG. 55.—*Beam 8AAD1; three-fourths inch through and one-half inch lapped bars. Sections I–K*

At a load of 15 000 pounds cracks showed considerable development. Horizontal cracks appeared along the outside tension bars at each end of the beam. No breaking of the bond at the observation points was observed at this load.

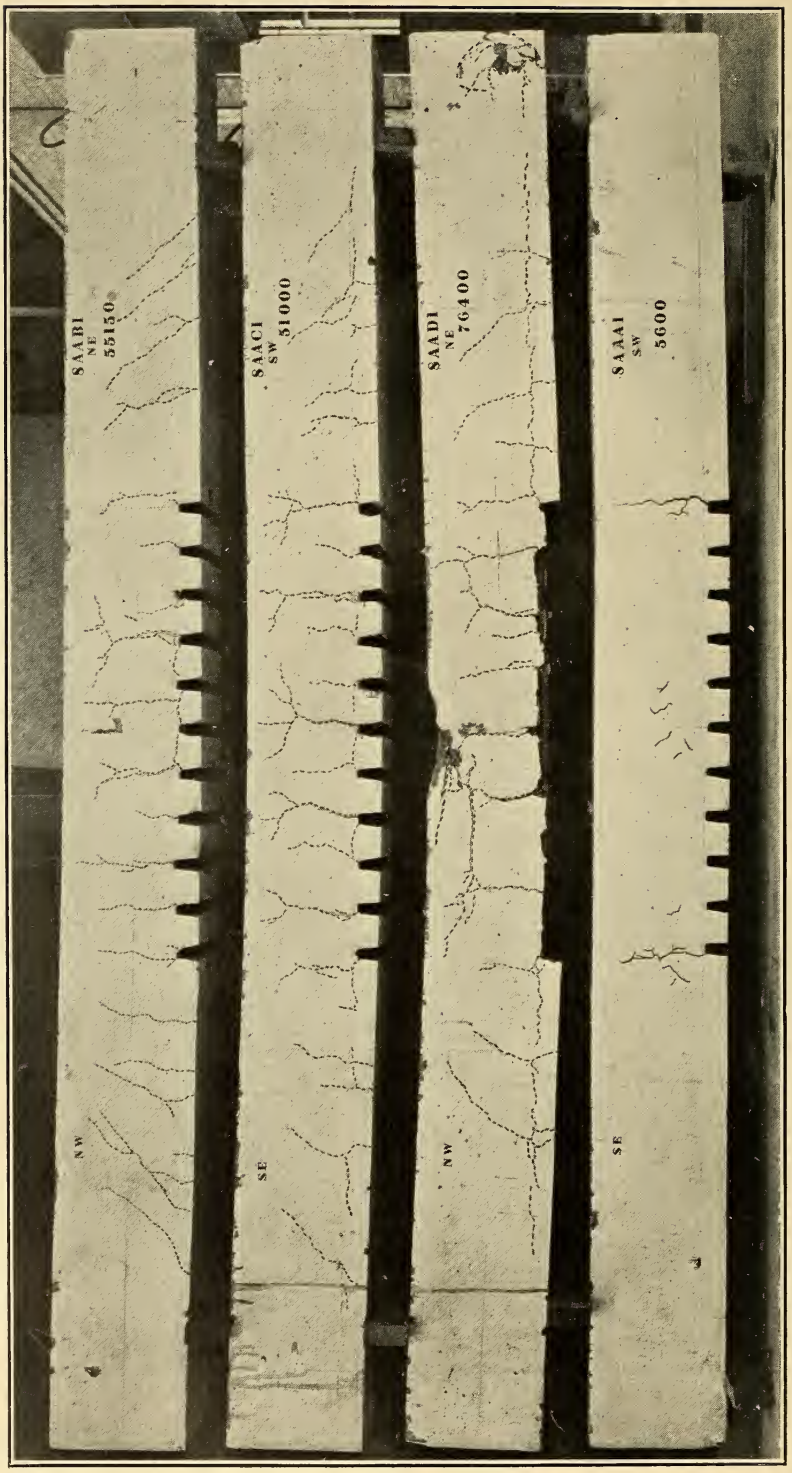


FIG. 56.—*Beams after failure, showing crack development*

At a load of 35 000 pounds diagonal tension cracks showed in all quarters, and tension cracks appeared at the center of the beam. The first two observation points at the free end of each of the lapped bars showed slip.

With a load of 50 000 pounds on the beam, the whitewashed observation points showed movement of the lapped bars for four consecutive points from the free ends of the bars.

At a load of 65 000 pounds slip was observed at seven consecutive points. No movement of the through bars was observed. While standing at the load of 65 000 pounds, two more observation points on bar No. 5 and one more on bar No. 4 showed movement.

At a load of 76 400 pounds cracks following the outer horizontal bars showed throughout the full length of the bars, following around the hooks at the southwest and northeast corners. The whitewashed observation points showed that the two outer through bars had slipped. One of the lapped bars showed slip at all the points of observation. The other bar showed slip at all but the observation point nearest the free end. The concrete started to split off at the southwest corner around the hooked end of the outer through bar. There was originally a three-thirty-seconds-inch covering of concrete over the bar at this point. This reading was taken at the maximum load. Although the shearing stress at this load was 465 pounds per square inch there was no indication that failure due to this cause was approaching.

The load dropped off to 73 000 pounds and remained at this point for some time with the machine running. The size of cracks indicated that the stress in the steel was reaching the yield point, and so a set of readings was taken. The readings at this load showed the steel to be overstressed in the locality of these cracks.

Loading was resumed at constant speed. The load fell off to 68 000 pounds and continued at this point for some time, or until a failure by compression of the concrete on the top of the beam at the center of the span occurred.

This failure was sudden and accompanied by a sharp report. On the northeast corner the concrete had spalled off in about the same manner as at the southwest corner, due to slipping of the hooked end of the outside bar. The other corners were not affected. The other three-fourths-inch bars did not show any failure of bond except at the region where the stress exceeded the yield point of the steel. All the points of observation on the lapped bars which were covered with whitewash showed movement. The movement of the ends of these bars was over one-half inch.

7. GENERAL RESULTS

Table 14 gives some of the results of these tests.

Fig. 56 shows a view of all the beams after test. The cracks are shown by broken lines. Fig. 63 gives the load-deflection diagrams for all the beams. Figs. 43 to 56 give load-stress diagrams for all the gage lines for various sections of the portions of the beams under uniform bending moment.

8. DISTRIBUTION OF TENSILE STRESS AMONG BARS

Figs. 57 to 62 give the intensity of tensile stress among the bars across the beams at various sections. Two loads are represented in each case. The average tensile stress for all bars at each load is shown by a broken line. A study of these diagrams shows that there was a point on each lapped bar beyond which there was a falling off of the stress due to the slipping of the free end of the bar. Proceeding from this point toward the end of the bar, the intensity of stress steadily decreased, and the stress in the other

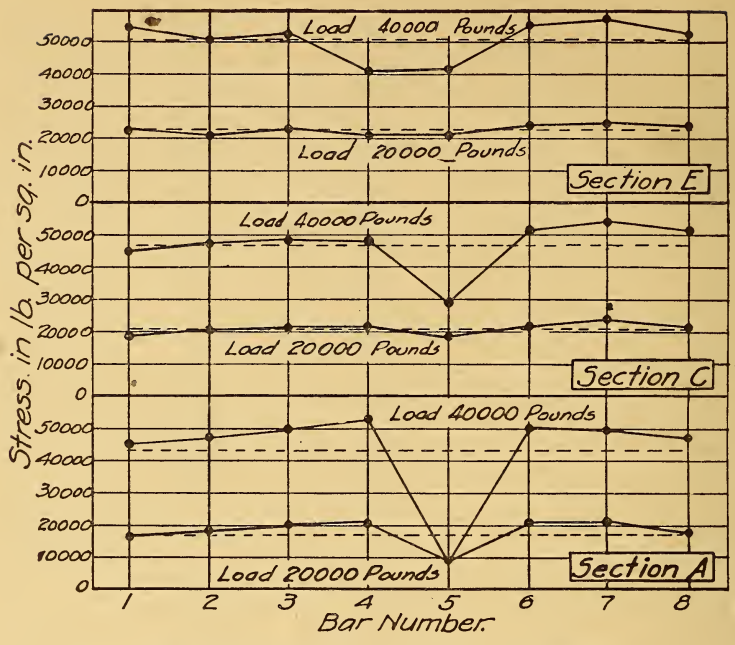


FIG. 57.—*Beam 8AABI. Sections A, C, and E*

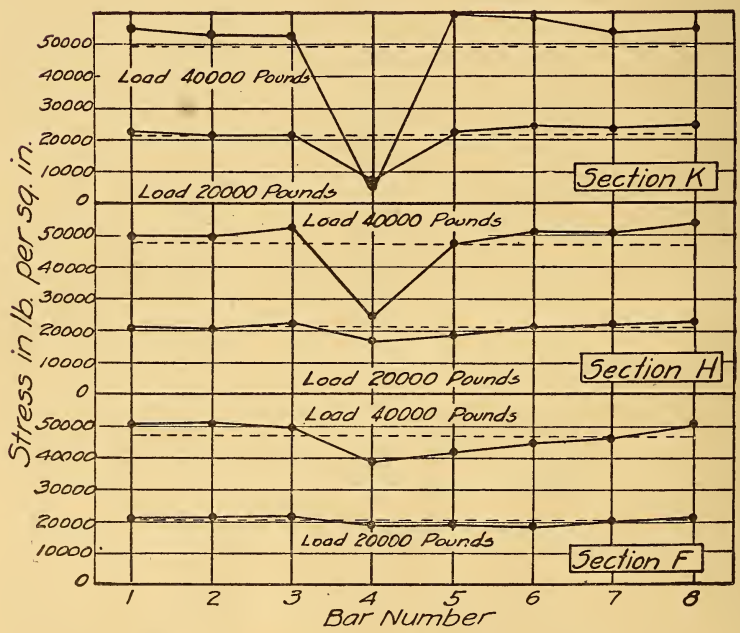


FIG. 58.—*Beam 8AABI. Sections F, H, and K*

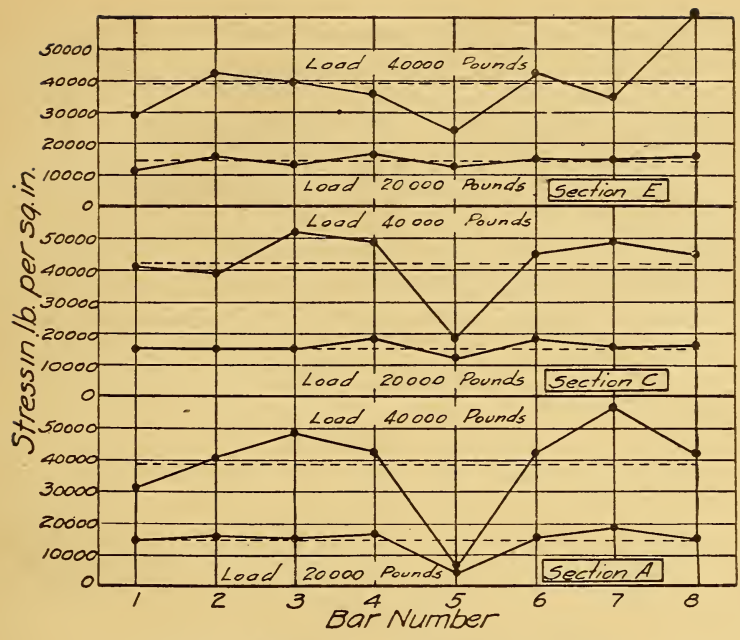


FIG. 59.—*Beam 8AAC1. Sections A, C, and E*

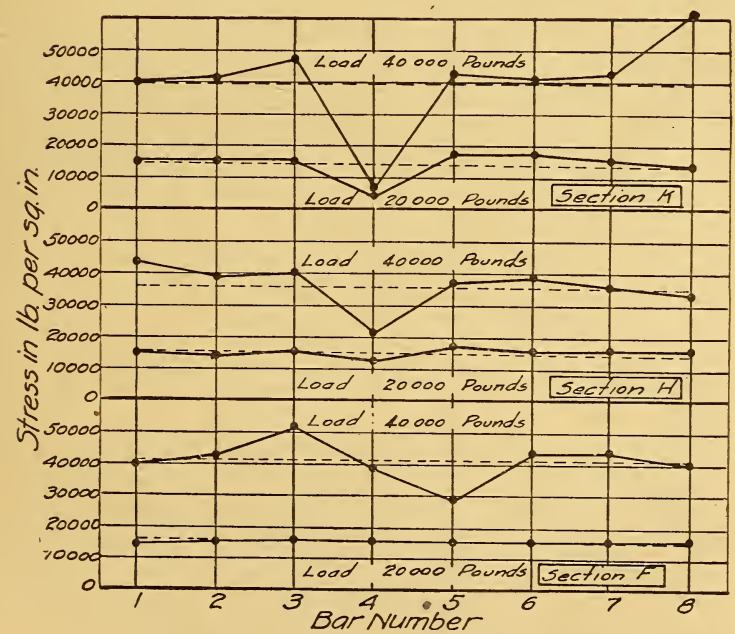


FIG. 60.—*Beam 8AAC1. Sections F, H, and K*

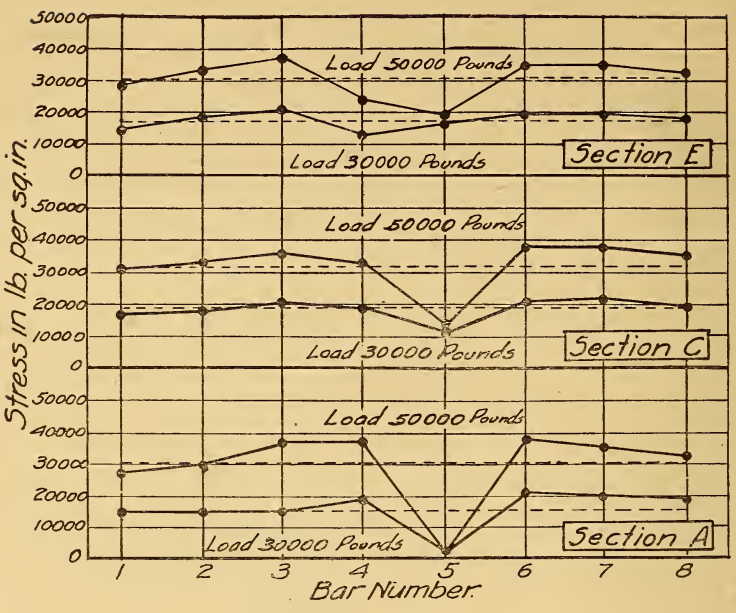


FIG. 61.—*Beam 8AAD1. Sections A, C, and E*

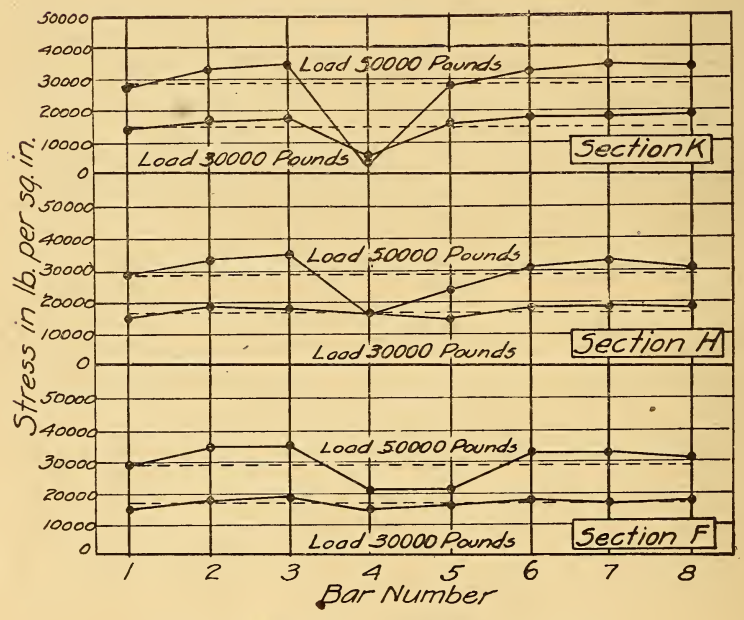


FIG. 62.—*Beam 8AAD1. Sections F, H, and K*

bars of the beams increased correspondingly at the same sections. In most cases the increase in stress was greater in the through bars immediately adjacent to the lapped bars than in the other bars of the beam. However, this tendency is not pronounced, and there is no evidence that it is affected by the variation in the proportional areas of through and of lapped bars.

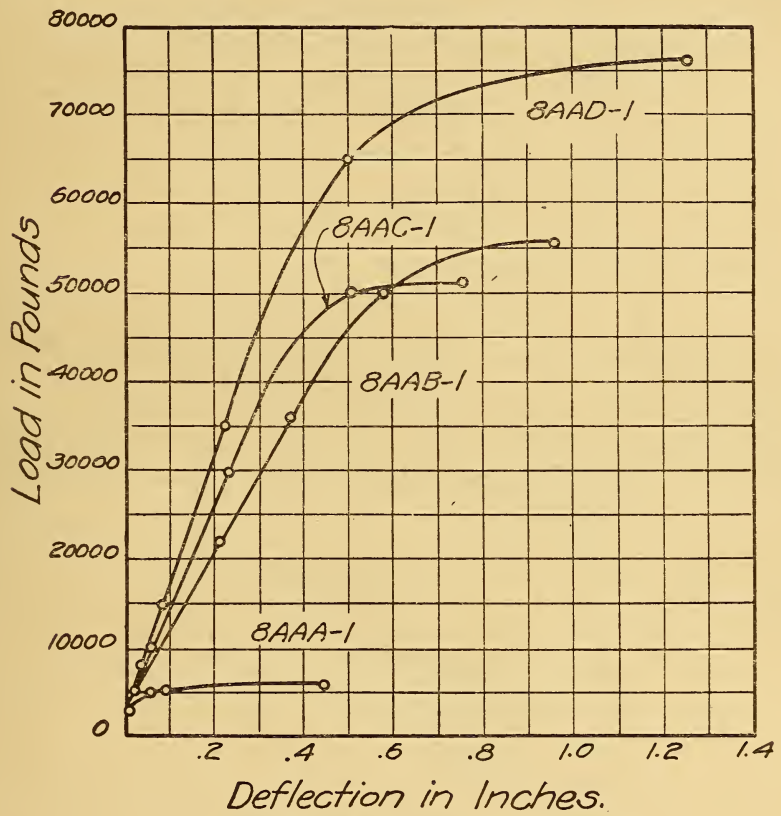


FIG. 63.—*Load-deflection diagrams for all beams*

9. DISTRIBUTION OF TENSILE STRESS ALONG BARS

Figs. 64 and 65 show the distribution of tensile stress as measured on the lapped bars for each of the beams. For the beams with through bars, the average tensile stress for all the through bars at the corresponding sections is shown by a broken line. Curves are given for loads giving computed stresses of 16 000 and 40 000 pounds per square inch in the tension bars.

The curve for beam 8AAD1, Fig. 65, at a computed stress of 40 000 pounds per square inch, shows that the stress in one of the bars of the lap did not become equal to that in the adjacent through bars within the length of the splice. This was probably

due to the porous condition of the original concrete around the tension reinforcement of this beam.

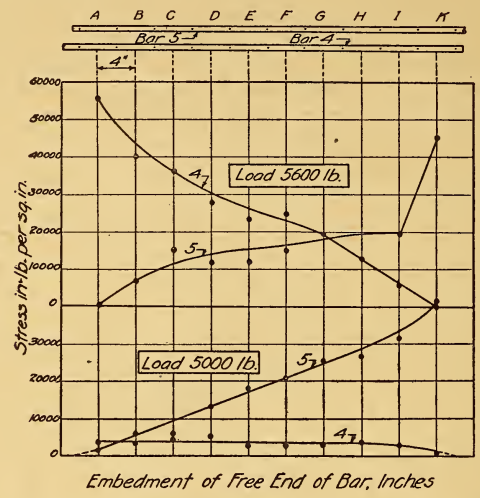


FIG. 64.—*Distribution of tensile stress along lapped bar in beam 8AAA1*

The curves for beam 8AAA1, Fig. 64, are for a higher computed stress than 40 000 pounds per square inch. At the load which gives 40 000 pounds per square inch computed stress, cracking had

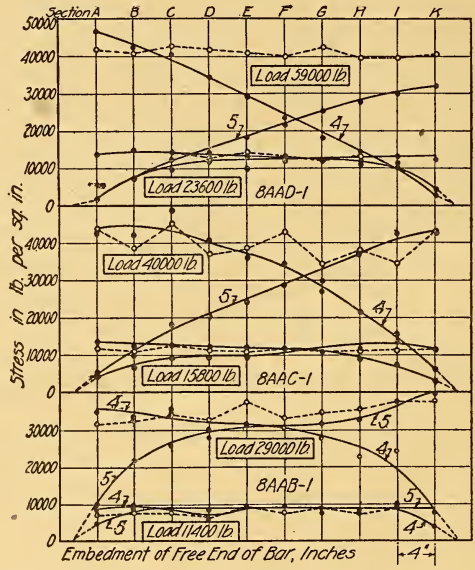


FIG. 65.—*Distribution of tensile stress along lapped bar in beams 8AAB1, 8AAC1, and 8AAD1*

not developed sufficiently to cause the tension reinforcement to carry all the tensile stress unaided by the concrete.

10. DISTRIBUTION OF BOND STRESS ALONG BARS

Figs. 66 and 67 show the distribution of bond stress along the bars of the lap for the three beams with through bars. Each curve is the average for the two lapped bars of a beam. The values plotted are proportional to the differences in tensile stress as measured by the strain gage on consecutive gage lines, starting at the free end of the bar. Curves for two loads are given. The curve for the lower load of each beam shows the distribution of bond stress along the bar when the bond stress at the free end of the bar is at or near the maximum. The curves for the higher loads

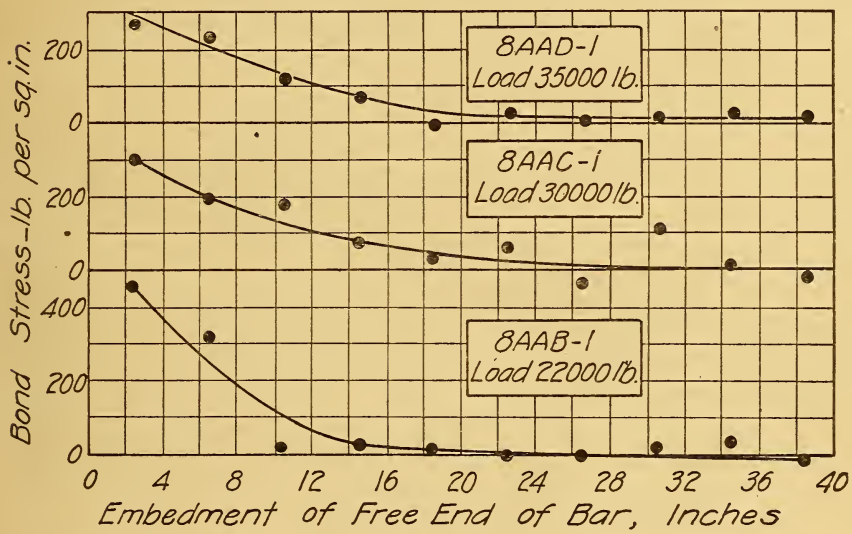


FIG. 66.—*Distribution of bond stress along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1*

have the same general form, but here the bond stress does not decrease to zero before the end of the lap of the adjacent bar is reached.

In the recorded bond stresses no correction has been made for the area of steel left exposed by the wooden blocks placed in the concrete for the purpose of locating the strain gage readings on the steel. This area would amount to about 5 per cent of the total surface area of the steel between gage points. Near the unanchored end of the lapped bars individual maximum bond stress values may be found which are about 75 per cent of the average values given in the first part of this report for pull-out tests of uncoated plain square bars.

11. SLIP OF BARS

Fig. 68 shows the progress of slip along the bars of the lap from the free ends to the ends of the splice for the same loads as those for which the bond-stress curves are given. Each curve is the average for the two lapped bars of a beam.

The slip for any point on the lapped bar at any load was obtained by subtracting the sum of the measured deformations between that point and a point where evidently no slip had oc-

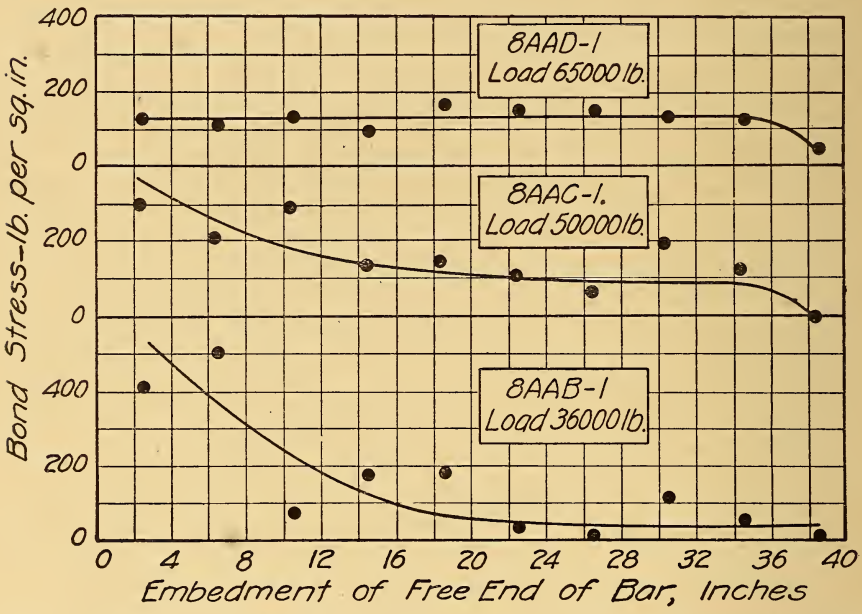


FIG. 67.—*Distribution of bond stress along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1 at higher loads*

curred from the sum of the corresponding deformations of the adjacent through bars. The original plan of measuring the slip of the end of the lapped bar was not used on account of the fact that in all cases a crack opened between the stub and the end of the bar. This caused an indeterminate error in the measurement of the slip, as taken by the strain gage, equal to the width of the crack.

A comparison of the bond-stress curves for the load which gives the maximum bond stress at the end of the lapped bar, Fig. 66, with the slip curves for the same loads, Fig. 68, shows that the maximum bond stress at the end of the bar occurred at a very small slip. However, the bond stress at the end of the bar fell off slowly from the maximum, and with a slip twice as great

as that which gave the maximum the bond stress was not very much reduced. This action is evidently due to frictional bond resistance. The slipping of the hooked bars of beam 8AAD1

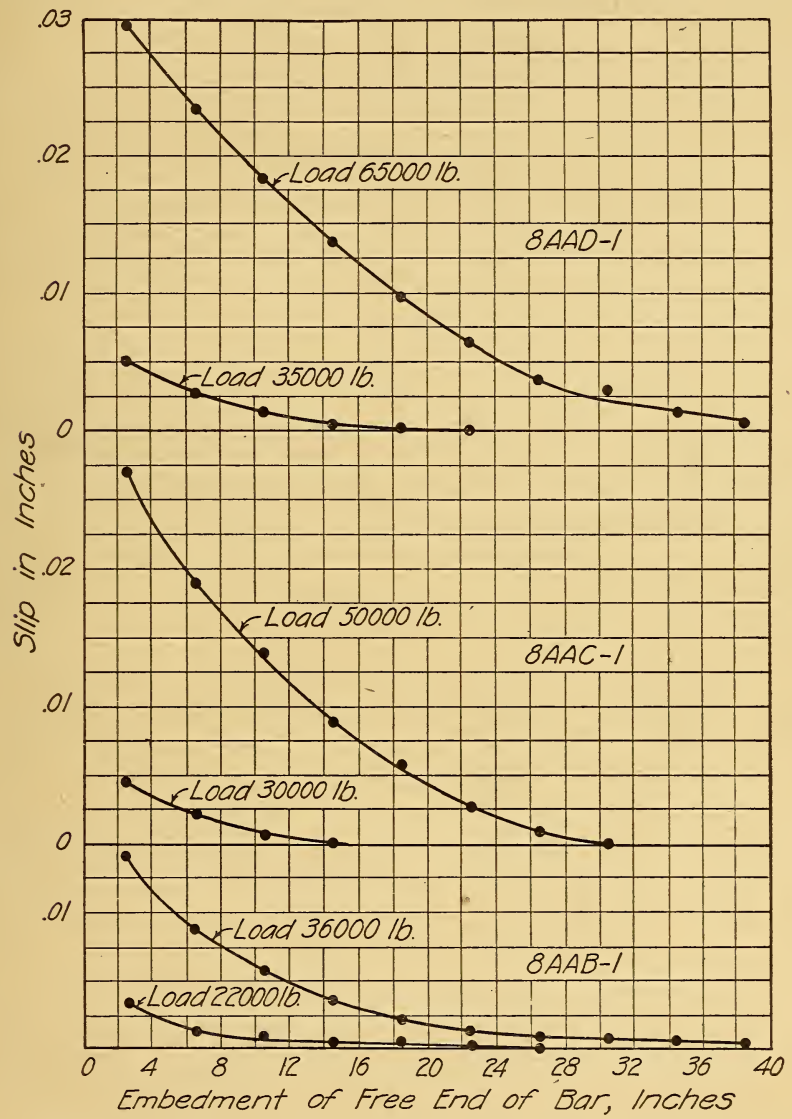


FIG. 68.—*Distribution of slip along lapped bars in beams 8AAB1, 8AAC1, and 8AAD1*

at the maximum load indicates that the bond resistance was insufficient to develop the full tension which existed in the bar and that the hooks were coming into action in furnishing anchorage. The computed bond stress at the maximum load was about

300 pounds per square inch or 6 per cent of the compressive strength of the concrete and the computed tensile stress was about 62 000 pounds per square inch. The lack of concrete in contact with the bars over a considerable distance probably contributed to this slipping. The condition of this beam is shown in Fig. 51. The same figure indicates that at least in the middle portion of the beam the outside bars were much closer to the sides of the beam than is called for in Fig. 39. If the condition at the end of the beam was similar to that at the center of the span, the bars were closer to the side of the beam than ordinary good practice would approve and this defect is probably responsible for the bursting of the hooked end of the bar through the side of the beam.

12. LENGTH OF LAP REQUIRED FOR BEAMS WITH THROUGH BARS

To serve as a basis for analysis, it may be assumed that for any given tensile stress the safe length of lap is the distance from the free end of the bar to the point where the tensile stress in the lapped bar becomes constant or equal to the stress in an adjacent through bar. If the lap furnished is less than this, the tensile stresses in the through bars at the ends of the lap must be greater than the value computed by the equation

$$f_s = M/A_s jd,$$

unless the difference in stress is taken up by the concrete in tension. This is based on the assumption that the entire lap lies within a region of constant bending moment.

With this assumption as a basis, Fig. 69 has been prepared. The data from the beam without through bars were not used in this figure. The amount of reinforcement for this beam was very small; consequently, the concrete took a large portion of the tensile stress, and the stress curves for the steel were too erratic for use in this way. The ordinates of the points given represent the distance from the ends of the lapped bars at which the measured stresses shown as abscissas were developed. A straight line represents fairly well the mean of the values shown by the points. The equation of this line is

$s = 0.00059\ f_s$, where (1)

s = distance in inches from end of lapped bar to a point where the bond stress becomes zero under a given load, and

f_s = the tensile unit stress computed from the corresponding bending moment.

It is possible that further investigation, using plain bars of diameters other than one-half inch, or deformed bars, would yield a different constant in this equation. A further modification would probably be produced by using concrete mixed in other proportions than 1 : 1 : 1.

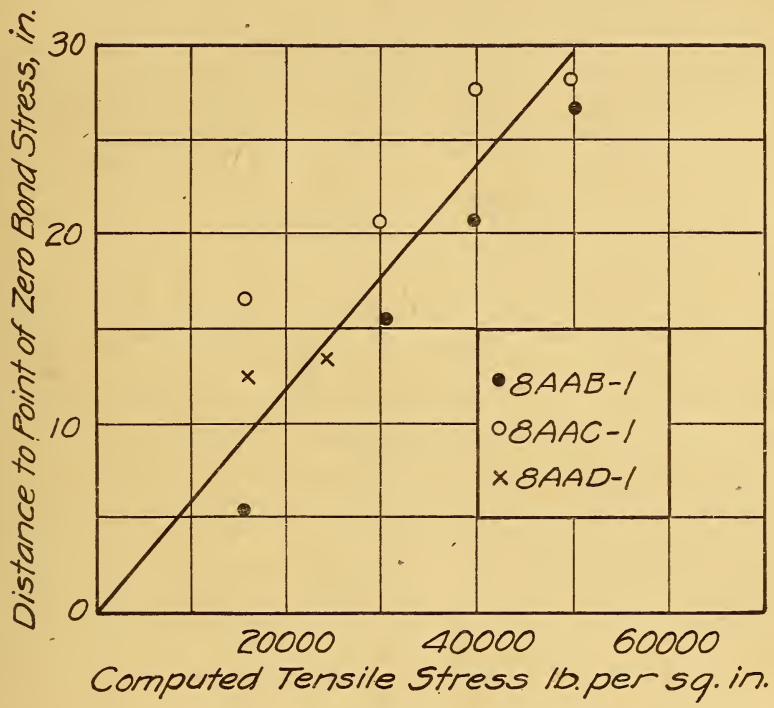


FIG. 69.—*Relation of tensile stress to length of lap required*

It is apparent that the total tensile stress in a lapped bar at the section where the bond stress becomes zero is

$$\frac{\pi a^2 f_s}{4} = \pi a u s, \text{ where} \qquad (2)$$

a = diameter of bar, and
u = average bond stress in distance s.

From this equation,

$$f_s = \frac{4\ us}{a}. \qquad (3)$$

Substituting in equation (1)

$$s = 0.00059\ \frac{4\ us}{a}, \text{ and} \qquad (4)$$

$$u = \frac{a}{4 \times 0.00059}. \qquad (5)$$

For lapped bars one-half inch in diameter the value of u from this equation equals 210 pounds per square inch. Equation (5) indicates that the value of the bond stress was independent of the tensile stress developed. The acceptance of this form of equation is equivalent to concluding that the average bond stress over the length s was the same at all stages of the test, but that the length s increased as the tensile stress increased.

Reference to beam 8AAB1, Figs. 66 and 67, shows that the distance s from the end of the bar at which the bond stress approximates zero was greater for the load of 36 000 pounds than for the load of 22 000 pounds, but that the average bond stress was not far from 210 pounds per square inch in both cases. For beams 8AAC1 and 8AAD1 the average bond stress was less than this. Apparently the conclusion that the average bond stress developed over the length of lap is independent of the tensile stress developed in the bars is not fully justified, but as a basis for design this assumption is probably exact enough.

It is generally assumed that the length of embedment required to develop the tensile strength of a bar is directly proportional to the diameter of the bar. In this investigation it was not feasible to test a sufficient number of beams with lapped bars to confirm this assumption, but it is here accepted as logically to be expected. Since these tests were made on one-half-inch bars, the equation becomes by substitution from equation (1),

$$\frac{s}{a}=\frac{0.00059}{\frac{1}{2}}f_s=0.00118\,f_s, \tag{6}$$

or,

$$s=0.00118\,f_s a. \tag{7}$$

The stress in the lapped bar just outside the lap is the f_s of this equation. If this is less than the average stress at this section calculated by the equation

$$f_s=\frac{M}{A_s\,jd},$$

then the stress in the through bars must be greater unless the difference is taken up by the tensile resistance of the concrete. Consequently, in using equation (7) to compute the minimum safe length of lap, the value of f_s should be the yield point of the steel. If f_s be taken less than this, the yield point will be passed in the through bars before it is reached in the lapped bars. Assuming the yield point to be 40 000 pounds per square inch, and that the concrete has the same strength as the concrete used in these tests,

$$s=47.2a. \tag{8}$$

Under a working load the total tensile stress developed in the lapped bar within the distance s from its free end is as stated in equation (2). In this case the values of u and s to be used are the working stress. From this equation, using $f_s = 16\,000$,

$$u = \frac{af_s}{4s} = 212 \text{ pounds per square inch.}$$

From Table 14 the average compressive strength of the control cylinders was 5020 pounds per square inch. Assuming that the bond resistance is proportional to the compressive strength of the concrete,[9] f'_c,

$$\frac{u}{f'_c} = \frac{85}{5020} = 0.0169, \text{ and}$$

$$u = 0.0169\, f'_c \qquad (9)$$

13. EFFECT OF VARIATION OF AREA OF THROUGH BARS ON LENGTH OF LAP REQUIRED

A study of Fig. 69 indicates that within the limits of the data used for this diagram a variation in the ratio of the area of the through bars to that of the lapped bars had little if any effect on the length of lap required. However, in these tests the least area of through bars was six times that of the lapped bars. With smaller proportions of through bars it is possible that a greater length of lap might have been needed. An extreme case is represented by beam 8AAA1, which had no through bars. This beam failed by the pulling out of one of the bars, although the lap was 80 diameters. This occurred at a load which gave a computed tensile stress in the steel of 50 000 pounds per square inch, and a measured stress of 55 000 pounds per square inch at one point. While this stress was high, it was less than the yield point of the steel. The evidence afforded by the failure of beam 8AAA1 indicates that a greater length of lap is required for beams in which all the bars are lapped than for beams in which there are through bars. Further investigation along this line is desirable.

14. SUMMARY OF RESULTS

The following statements summarize the results of the beam tests reported above:

(*a*) Proceeding along a lapped bar toward its unanchored end from the point where the stress in it begins to diminish, the stress lost is picked up by the other bars of the beam. In a majority of

[9] Abrams, Tests of Bond Between Steel and Concrete, Bull. 71, Eng. Exp. Sta., Univ. of Illinois, p. 94, 1913.

cases a larger amount of the stress lost by the lapped bar was picked up by bars immediately adjacent than by the other bars of the beam.

(*b*) The maximum bond stress developed by the lapped bars was about 75 per cent of that reported for pull-out tests of uncoated plain square bars in Division II of this paper. The maximum bond stress occurred at a smaller amount of slip of bar for the beams than for the pull-out tests.

(*c*) The average bond stress in the lapped bars was practically independent of the tensile stress in the bars, but the length over which it was effective varied with the magnitude of the stress.

(*d*) For the purpose of design the minimum safe length of lap may be taken as the distance from the unanchored end of the lapped bar to a point on the bar where the bond stress is zero, when the tensile stress in the steel is at the yield point. With this assumption the tests indicate that using steel which has a yield point stress of 40 000 pounds per square inch, and with continuous bars adjacent to the lapped bars and with concrete of the grade here used, the lap should be about 48 diameters. Care should be used in applying this relation under other conditions.

(*e*) The tests of beams with through bars do not show that differences in the ratio of the area of the through bars to the total area cause any variation in the length of lap required. The test of the beam with no through bars indicated that when all the bars are lapped, a longer lap is needed than when through bars are present. Further investigation is needed on this phase of the subject.

(*f*) A shearing stress of 465 pounds per square inch was developed in one of the beams without sign of approaching failure by diagonal tension.

IV. TESTS ON STIRRUP PULL-OUT SPECIMENS

1. PRELIMINARY

The tests described in the following pages of this paper were made on 12 test specimens designed to furnish useful information on the strength of stirrups. The dimensions of the specimens and reinforcement used conformed to a typical section through the shell and frame in a concrete oil-tank ship which was built by the Emergency Fleet Corporation. In such a section, due to greater stiffness in the frame than in the shell, the hydrostatic pressure of the cargo tends to force the shell away from the frame, thus

adding direct tension to the tension induced in the stirrups by shearing stresses in the frame. Under such circumstances the efficiency of the anchorage of the stirrups is of great importance.

2. DESCRIPTION OF SPECIMENS

The specimen consisted of a concrete slab 4 inches thick, cast monolithic, with a concrete rib 6½ inches wide and 12 inches deep. The slab was 1 foot 10½ inches long and 1 foot 4 inches wide. The rib was 2 feet 4 inches long and was attached to the slab along its center line in the short direction; it projected 6

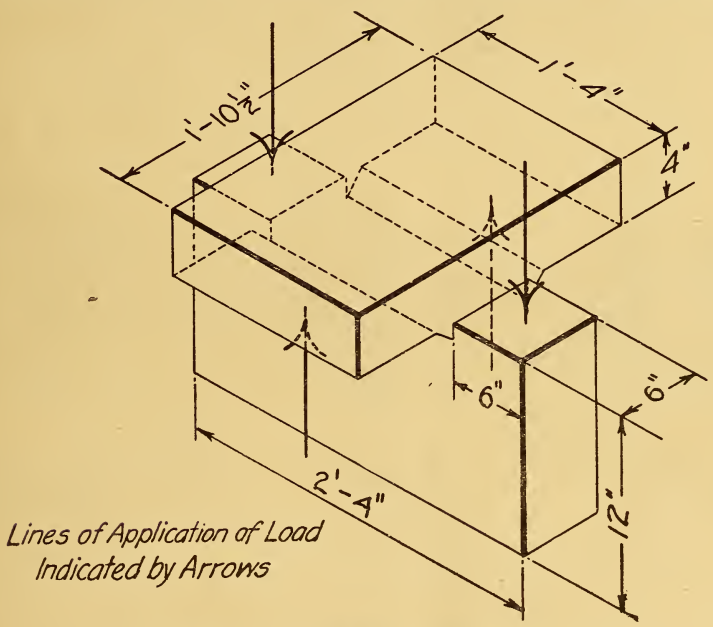


FIG. 70.—*Isometric view of specimen used in stirrup pull-out tests*

inches beyond the slab at each end to afford a bearing for the downward load. Fig. 70 is an isometric view of the test specimen.

The reinforcement in the prism and in the slab was placed so as to make the specimen as much like the corresponding section of the ship as possible. The three bars placed at the junction of the prism and the slab made the section weak in resistance to a tensile stress tending to separate the slab from the rib. The specimens were designated by letters from A to L, inclusive.

Three types of stirrups were used, as shown in Fig. 71, and designated as Types I, II, and III, respectively. The stirrups were made from one-half-inch plain round steel bars, bent into U shape with hooked ends. Type I stirrups had the hooked ends

bent perpendicular to the plane of the legs and were used in specimens D, E, F, K, and L. Type II stirrups had the ends bent into loops through an angle of 270°, in which the planes of the loops were perpendicular to the plane of the legs. Stirrups of Type II were used in specimens A, B, C, G, H, and I. Type III stirrups had the ends bent outward at 90° from the legs and in the same plane. This type was used in specimen J.

Two stirrups of the same type were used in each specimen. They were placed 6 inches apart. The hooked ends of the stirrups were arranged at different depths in the slab member of the specimens and, in consequence, varied in relation to the position of the slab reinforcement. The distance between the hooks and lower surface of the slab for each specimen is shown in Table 15. The loops in the stirrups used in specimens G, H, and I were

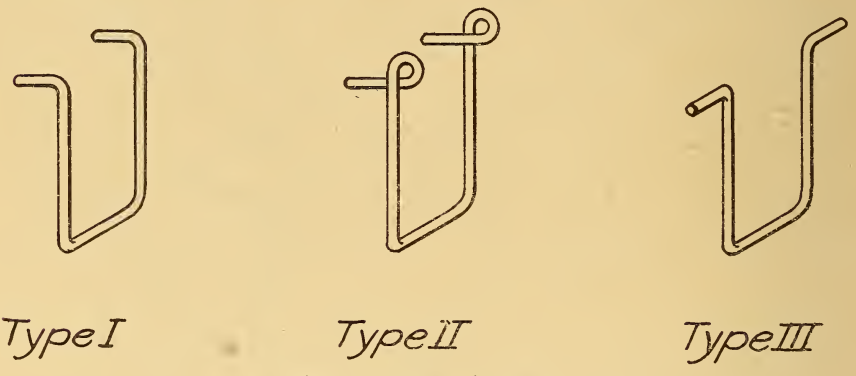


FIG. 71.—*Types of stirrups*

stuffed with mineral wool in order to exclude concrete during the pouring and to produce the condition of a void within the loop, a condition which is sometimes found in actual construction.

Fig. 72 shows the elevation and cross section of a typical specimen with the reinforcement indicated.

3. MAKING OF SPECIMENS

The concrete was mixed in the proportions 1:⅔:1⅓, measured by volume. Lehigh Portland cement and burnt-clay aggregates manufactured by the Atlas Portland Cement Co. were used. The consistency of the concrete was determined for each batch by the slump test; the average drop in a 6 by 12 inch cylinder was 9 inches when the mold was lifted. The concrete was representative of that used in the construction of concrete ships. In specimens A to F, inclusive, it was necessary to substitute sand and gravel

in the concrete used to a depth of about 6 inches in the rib of the specimen. This change was required on account of a shortage in the amount of burnt-clay aggregates available. The results of the tests show no effect that may be attributed to this change in mix. The specimens were poured from the top of the slab after the reinforcement was wired in position.

4. AUXILIARY SPECIMENS

To determine the strength of the concrete at the same ages as the stirrup pull-out specimens when they were tested, 19 control cylinders, 3 by 6 inches in size, were molded from the batches mixed for the pull-out specimens. The average results of the

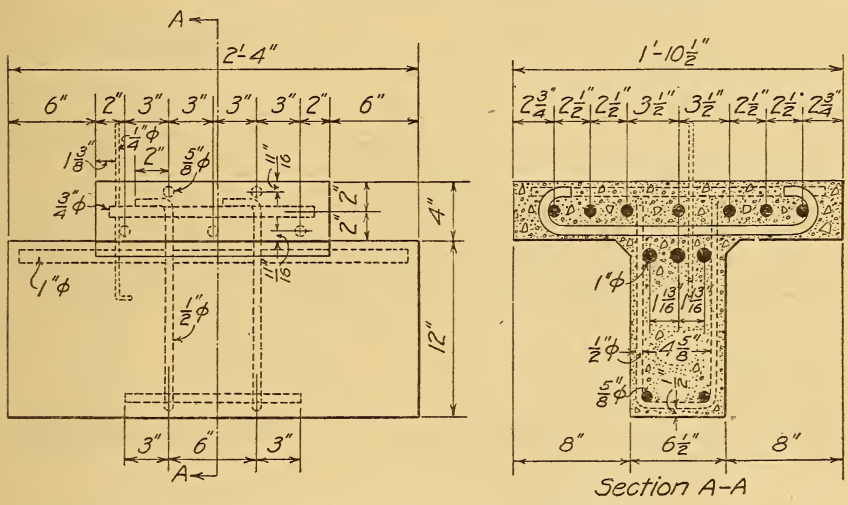


FIG. 72.—*Details of stirrup pull-out specimen*

compression tests on these cylinders are given in Table 15 and show a range of strength from 3900 to 4900 pounds per square inch.

5. METHOD OF TESTING

The stirrup pull-out specimens were tested in an Olsen testing machine of 200 000 pounds capacity. Fig. 73 shows a specimen arranged for testing.

A one-fourth inch plain round bar extended vertically through the slab of each specimen and 4 inches down into the rib. The bar was located 1⅜ inches from one edge of the slab. The lower end was anchored in the rib by means of a short L hook; the other end projected 4 inches above the slab. The upper half of the embedded length, or that passing through the slab member,

was coated with cup grease to destroy the bond resistance between the concrete and the rod. The purpose of this rod was to hold an Ames dial gage in contact with the slab during the test in order to determine, if possible, the instant when separation between the slab and the rib started.

The manner of loading the specimen was such as to pull the slab away from the rib. The overhanging ends of the slab member were supported on bearing plates and steel rollers by I beams, which were laid on the weighing table of the testing machine. The load was applied by the downward movement of the pulling head of the machine acting through a steel beam and spherical

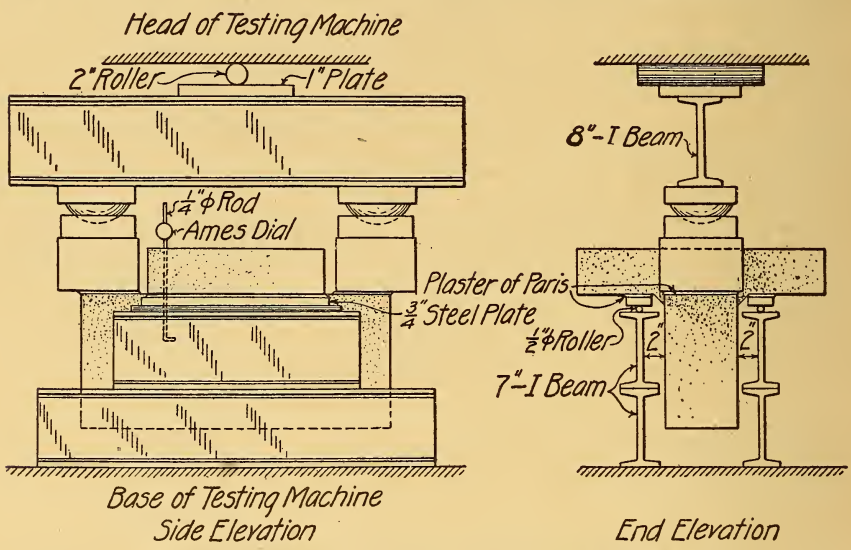


FIG. 73.—*Arrangement for testing stirrup pull-out specimen*

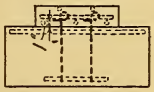
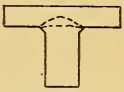
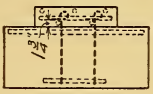
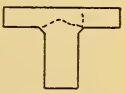
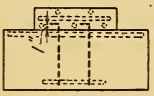
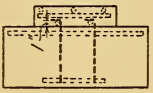
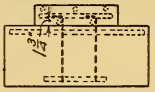
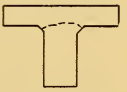
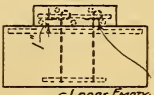
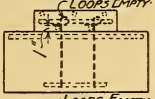
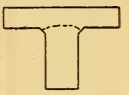
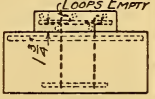
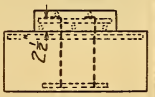
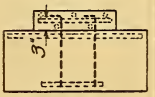
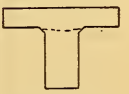
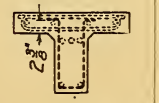
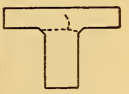
bearing blocks on the ends of the rib member of the specimen. The machine was stopped for examination of the specimen after each 3000-pound increment of load was applied.

The first indication of impending failure came by the sudden appearance of cracks in the plane at the junction of the slab and the rib of the specimen. Use of the Ames gage was abandoned after testing two or three specimens, as it gave no indications of value prior to failure.

6. RESULTS OF TESTS

Table 15 shows that the first crack occurred at about the same total load in all of the specimens except B and C, in which the load was somewhat higher. The sketches in Table 15 show that

TABLE 15.—Tests of Stirrup Pull-Out Specimens

Specimen and type	Age	Compressive strength of concrete	Load at first crack	Computed tensile stress in stirrups at first crack	Maximum load	Computed tensile stress in stirrups at maximum load	View showing position of stirrups	End view showing first crack
	Days.	Lbs./in.²	Pounds.	Lbs./in.²	Pounds.	Lbs./in.²		
A–II	31	4000	14 000	17 800	22 600	28 800		
B–II	36	4200	19 000	24 200	23 500	29 900		
C–II	31	4000	18 000	22 900	32 700	41 600		
D–I	32	4100	14 000	17 800	15 300	19 500		
E–I	31	4000	12 500	15 900	12 500	15 900		
F–I	30	3900	15 000	19 100	19 500	24 800		
G–II	17	4900	15 000	19 100	21 700	27 600		
H–II	18	4900	13 500	17 200	19 000	24 200		
I–II	12	4500	15 000	19 100	18 500	23 600		
J–I	14	4600	16 500	21 000	25 000	31 800		
K–I	17	4900	14 000	17 800	16 000	20 400		
L–III	13	4600	12 000	15 300	19 000	24 200		

for most of the specimens the surface of rupture was substantially a plane coinciding with the lower surface of the slab. They indicate that for specimens B and C the surface of rupture had an area somewhat greater than that for the other specimens. If this could be taken into account in the calculations, the tensile stress at the time of the formation of the first crack might be found to be nearly the same for all specimens. Immediately after the failure of the concrete the entire load on the specimen was transferred to the stirrups. The position of the surface of rupture caused by the first crack had an important bearing upon the total resistance of the stirrup anchorage, since in many cases it passed so close to the hooks as to leave them little covering of concrete.

The tests show that Type II stirrups were much more effective when the loops were filled with concrete than when they were empty, and under such conditions were stronger than either Type I or Type III. Even when the loops were not filled with concrete the stirrups of Type II were better on the average than those of Type I. The one specimen having Type III stirrups gave greater load at the first crack and greater maximum load than any of the specimens having stirrups of Type I. The first crack probably had less effect on the embedment of the hooks in the Type III stirrups than in the stirrups of Type I.

In general, the stirrups which had the deepest embedment of the hooked and looped ends showed the greatest strength. An embedment of not less than 2 inches is apparently desirable.

Comparison of specimen D with E and G with H indicates that no increase in effectiveness was obtained by having the stirrup hooks rest on a bar of steel in the slab reinforcement.

In making comparisons consideration should be given to the difference in age of the specimens when tested.

The appearance of the specimens after failure is shown in Figs. 74 to 85, inclusive.

WASHINGTON, April 28, 1920.

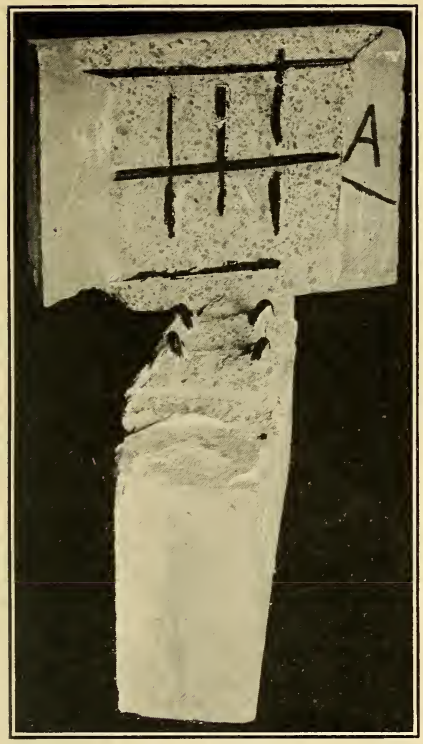


FIG. 74.—*View of specimen A after failure*

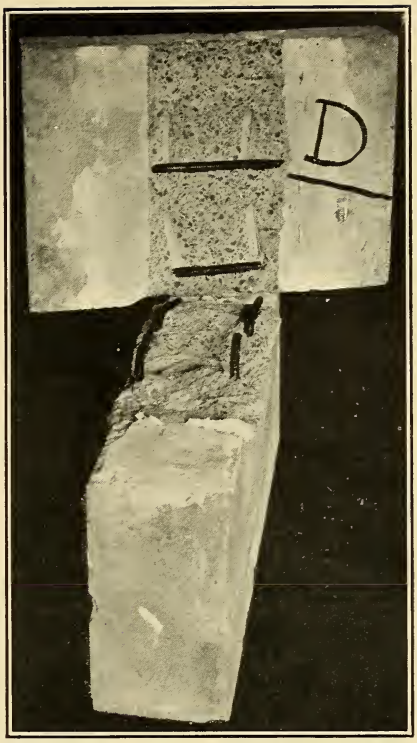


FIG. 75.—*View of specimen D after failure*

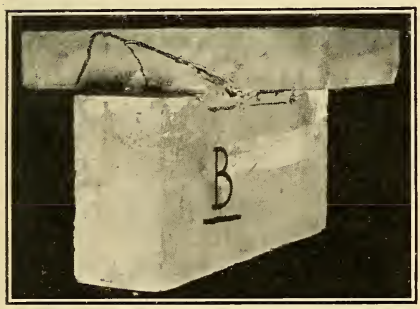


FIG. 76.—*View of specimen B after failure*

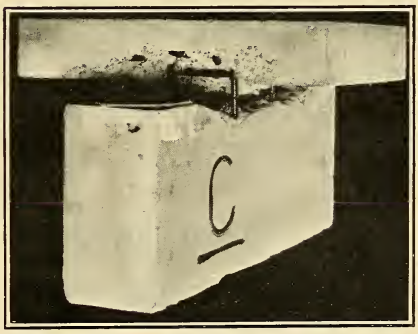


FIG. 77.—*View of specimen C after failure*

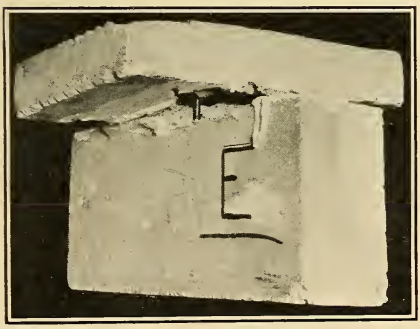


FIG. 78.—*View of specimen E after failure*

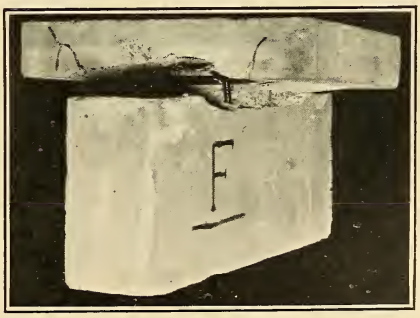


FIG. 79.—*View of specimen F after failure*

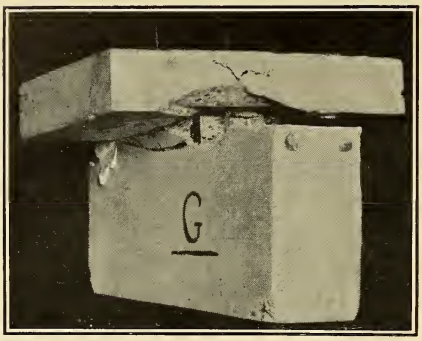


FIG. 80.—*View of specimen G after failure*

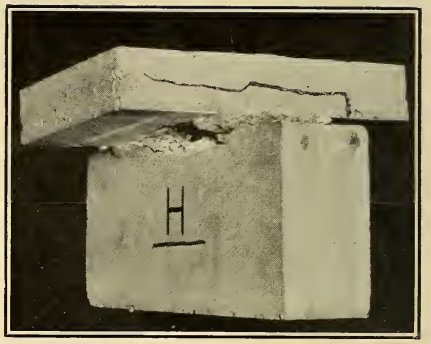


FIG. 81.—*View of specimen H after failure*

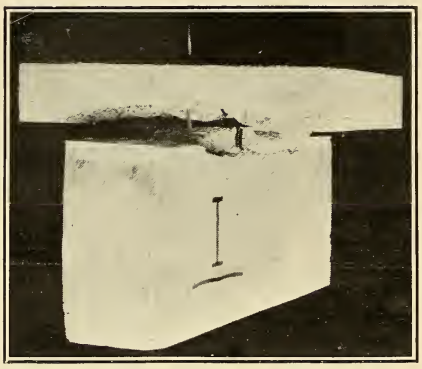


FIG. 82.—*View of specimen I after failure*

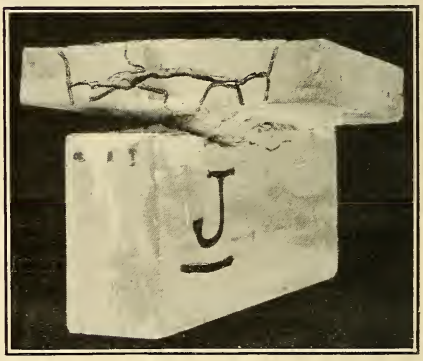


FIG. 83.—*View of specimen J after failure*

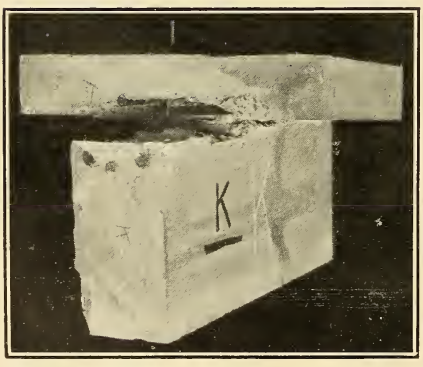


FIG. 84.—*View of specimen K after failure*

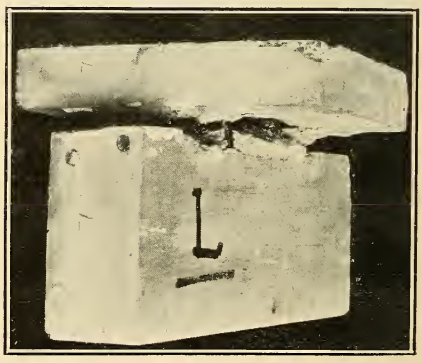


FIG. 85.—*View of specimen L after failure*